JOURNAL of the

Hydraulics

Division

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager, Technical Publications, ASCE. Authors should indicate the technical division to which the paper should be referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedure by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, ¼-in. high (with all other symbols and characters, in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller on the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper with a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 60% of the original size, the capital letters should be ¼-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Specific reference and descriptions should be made within the text for each illustration.

8. Papers should average about 11,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

April, 1959

Papers

fac
dra
anc
col

rel
ing
sig

the
rep
Un
ava
tin

Th
yea
wa
out
siz
170
dra
100

sir

No

1.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CHANNEL-SLOPE FACTOR IN FLOOD-FREQUENCY ANALYSIS

Manuel A. Benson,[1] A.M. ASCE

ABSTRACT

Annual flood peaks in New England have been related to many hydrologic factors. The main-channel slope has been found next in importance to drainage-area size. The slope for that part of the main channel between 85 and 10 percent of the total distance above the gaging point provides the best correlation with flood magnitudes.

Many agencies and individuals are studying the causes of floods and their relation to known meteorologic and physiographic conditions, and are developing generalized flood-frequency relations for the purposes of hydraulic design, flood insurance, flood-plain occupancy, and other uses.

Data for New England have been used in the first phase of such a study by the U. S. Geological Survey. The records of runoff and precipitation for this region are as numerous and as long as records for any other region in the United States; topographic quadrangle maps covering the entire area are available; historical flood data are available, dating back to the earliest times of settlement.

Peak discharge records of 170 gaging stations were used in the analysis. These records represented all the stations in New England having at least 10 years of record, on which the effect of regulation on annual peak discharges was known, or judged, to be negligible. These stations were located throughout the region, though not as densely in the northern part. The range in the size of drainage areas was from 1.64 to 9,661 square miles. Of a total of 170 station, 3 stations drained areas less than 10 square miles; 31 stations drained areas less than 50 square miles; 65 stations drained areas less than 100 square miles; and 106 stations drained areas less than 200 square miles.

In practice, there are many ways of converting a set of flood data at a single station to a flood-frequency curve or relation for that station, to

determine the floods of various recurrence intervals. It was considered that
the use of mathematically fitted curves involving prior assumptions as to
distribution might obscure the very relationships that were being sought, and
that graphically drawn curves, conforming as closely as possible to the
original data at each station, would be most desirable. The plotting position
of each flood of record was computed by using the formula $\frac{n+1}{m}$, where n is
the number of years of record and m is the rank of the flood, starting with 1
as the highest. This formula has theoretical basis and also agrees closely
with the length of the period of record for the top flood, thus again conform-
ing closely to the basic data.

From the frequency curves, values were selected for the 1.2-, 2-, 5-, 10-,
and 25-year floods at 170 stations, and for the 50-year flood at 40 stations.
Where the length of record was less than 25 years, correlation with nearby
stations was used to ascertain the appropriate plotting positions within a 25-
year period for the annual peaks of record. The floods for each of the stated
recurrence intervals were then separately correlated with a large number of
physiographic and meteorologic factors. Initially, the relations were ana-
lyzed by graphical multiple-correlation techniques combined with rank-
correlation methods. The following parameters were studied:

1. Drainage area

2. Slope
   - Channel slope No. 1
   - Channel slope No. 2
   - Average land slope
   - Tributary channel slope

3. Water-supply index
   - Mean annual runoff
   - Mean annual precipitation
   - Maximum 24-hour precipitation
   - 10-year 60-minute intensity
   - Total rainfall volume in 6 large storms

4. Shape factor
   - L^2/A, equivalent of basin length divided by basin width
   - L/A, basin length divided by drainage area
   - Σal, the summation of small subdivisions of the drainage area, each
     multiplied by the distance of travel to the gaging point.

5. Mean elevation

6. Percentage of lakes, swamps and reservoirs

7. Stream density

8. Temperature (January average)

Most of the physical characteristics were computed as described by
Langbein [1947]. "Channel slope No. 1" is one such characteristic. It is the
mean slope of all parts of a channel draining at least 10 percent of the total
drainage area,—thus it may include parts of the principal tributary streams.
"Channel slope No. 2" is the same as that described by Bigwood and Thomas
[1955]; the weighted main-channel slope is averaged with the tributary
channel slopes as described by these authors.

The mean annual precipitation for each drainage area was supplied by the U. S. Weather Bureau as computed in the preparation of Hydrologic Atlas HA 7 [Knox and Nordensen]. Other precipitation factors were based on various Weather Bureau publications or on Geological Survey flood reports.

Many of the hydrologic factors, such as "channel slope No. 1" or "stream density" were available from Water Supply Paper 968-C, Langbein, [1947], though not for all the 170 stations usable in New England. Because of the labor involved in computing these, it was considered satisfactory, in the preliminary phases of the study, to compare the various factors and test their significance using only the 85 to 95 stations for which most of the hydrologic factors were already evaluated.

In the graphical procedure used, Q_n, the n-year flood, was first plotted (on log-log paper) against the drainage area. Through the plotted points, an average curve was drawn, and the residuals from this curve were determined for each station and ranked in order of magnitude. The values for all other parameters were then ranked and the differences between ranks were used to compute the Spearman rank correlation coefficients [Wallis and Roberts, 1956]. The parameter with the highest correlation (considering only one from each numbered group in the preceding table) was then used as the next variable in the graphical correlation. After the proper adjustments, a plot of (adjusted) first residuals versus the first parameter values led to a set of secondary residuals, which was in turn tested against the remaining parameters.

Table 1 shows the results of graphical correlation for the 2-, 10-, and 25-year floods for all significant parameters. This shows the decrease in the standard error when successive independent variables were included. In each case, after 3 or 4 independent variables were used, further variables did not appreciably decrease the standard error. It is notable that factors evaluating tributary slope alone were not found to be significant, once the effect of the main channel slope, as expressed in channel-slope factor no. 1, was accounted for. The graphical standard errors were determined as the range within which two-thirds of the plotted points lie. Actually these are not standard errors of estimate, but are approximations adequate for these exploratory studies. The standard errors are shown in terms of the original log units and in the equivalent percentages these represent. One log unit equals one log cycle and a decimal part of a log unit is that same decimal part of the linear distance representing one log cycle.

As would be expected, the standard error increases with the increase in the size of flood. It will be noted that for each size of flood, the standard error with drainage area (A) drops sharply after channel slope (S) is included, and is improved by only a small amount thereafter. The average percentage improvement with S is 22, 38, and 19 percent respectively, whereas the other parameters after A and S show combined improvements not exceeding 6 percent.

It will be noted also that water supply indices, R, P, and I, show but small effect. The relatively minor influence of rainfall is attributed to the uniform pattern of precipitation throughout New England and to the unsatisfactory rainfall parameters so far available.

The importance of channel slope is shown by the large percentage of the error which it accounts for. However there has been no unique or universally accepted way of evaluating channel slope. Some hydrologists have used the total drop from the head of the longest watercourse. Others have used

Table 1.--Results of Graphical Multiple Correlation

Q_2	log units	percent		ave
		+	−	
A	0.24	74	42	58
A, S	.155	43	30	36
A, S, R	.15	41	29	35
A, S, R, L/A	.13	35	25	30
Q_{10}				
A	.31	104	51	78
A, S	.17	48	32	40
A, S, P	.152	42	29	36
Q_{25}				
A	.26	82	45	64
A, S	.19	55	35	45
A, S, I	.17	48	32	40
A, S, I, L/A	.165	47	31	39

A=Drainage area; S=Channel slope No. 1; R=Mean annual runoff;
P=Mean annual precipitation; I=10-year, 60-minute maximum
precipitation; L/A=Shape factor.

weighting methods which evaluate the slope all along the main channel. Some
have used parts of the main channel, such as the lower three-quarters. Still
others have in some manner combined the slopes of tributary streams with
the main channel. Part of the difficulty is that methods of defining the main
channel or of differentiating the tributary streams are not fixed.

Channel slope No. 1 was found to be the best slope factor of those evalu-
ated. Its computation involves the determination of the points on the larger
streams at which 10 percent of the total area is drained. This is a laborious
task, particularly in a regional study where it may have to be done for
hundreds of stations. It was therefore considered desirable to attempt to find
a simpler, and possibly better channel slope factor. It was also considered
desirable to separate the effects of tributary streams from those of the main
stream. It was decided (1) to use only the main channel in a channel-slope
parameter, leaving the tributary slopes for separate consideration, (2) to de-
fine the main channel, above each stream junction, as that channel draining
the largest area, (3) to make an exhaustive study to find a way of expressing
the main-channel slope that is most closely related to peak discharge.

Topographic maps were used to develop channel profiles for all 170 sta-
tions in New England. At the upstream end each profile was extended to the
drainage divide beyond the end of the stream shown on the topographic map.
Distances were measured from the gaging point to each contour crossing (ex-
cept where these were too dense) and channel profiles were plotted from
these figures.

It was considered that the most upstream part of the stream, in the steep headwaters, might affect the slope out of proportion to the volume of water furnished by the headwater area. At the downstream end the slope might not be indicative of that affecting the size of peak discharges because gaging sites are usually located at riffles or natural controls. It was therefore postulated that the part of the main channel whose slope would best correlate with peak discharge would be the one excluding the extreme headwater reach, and possibly some of the extreme downstream end.

The total lengths of the streams were divided into tenths below 0.7 of the distance from the gage and half-tenths beyond that. The object was to compute slopes for all possible portions of the channel, for example, 0.95 to 0.0, 0.7 to 0.1, 0.6 to 0.4, etc., the figures representing in each case the fraction of the total distance upstream from the gage. There were 91 such combinations. Two additional slope factors were computed. These were the constants in the regression equation relating the logarithm of the rise to the logarithm of the distance from the gage, equivalent to drawing a straight line through the channel profile plotted to a logarithmic scale. These two factors represent an integrated slope for the entire main channel, rather than merely the slope between two points.

To determine the optimum slope factor, multiple correlation analyses were to be made with the 1.2-, 2.33-, 5-, 10-, 25-, and 50-year floods as dependent variables and drainage area and each of the 93 slope factors as the independent variables. The change from the 2-year to the 2.33-year flood was made at this time in order to use a value corresponding to the mean annual flood in common use; results are comparable. The best slope factor would be defined at the point of minimum standard error. The correlation analyses were such a prodigious task that the data were prepared for automatic solution by a digital computer, the Datatron 205. The programming covered computation of all the slopes, the logarithmic slope factors, the standard errors with drainage areas alone, and the standard errors and correlation coefficients with drainage and each of the 93 slope factors. Data for 170 stations was used in these correlation studies.

Results of the Datatron solutions are shown in Figures 1 and 2. These figures show contours representing equal values of the standard error. On Figure 1, for example, the standard error using the slope between points 0.5 and 0.4 of the total distance above the gage is 0.193 log units. Figure 1 shows all the standard errors on which the contours are based whereas Figure 2 shows only the contours. Note the similar patterns shown by the controus for each size of flood and the minimum in the upper left of each figure. A weighting process reveals that the slope between points 85 and 10 percent above the gage would satisfactorily give the minimum standard error for all floods, with little accuracy lost for any particular size of flood. Standard errors using the two logarithmic slope factors were higher for all floods than those using the 85 to 10 percent slope.

Table 2 shows the standard errors (in log units and average percent) for each size of flood, using drainage area and the "85-10" percent slope as independent variables. The improvement in each can be seen when slope is introduced. A regularity shows up in the percentage of the error explained by slope, which is close to 35 percent in each case. The standard error for the 50-year flood seems inconsistently small and the contour pattern is somewhat erratic. The irregularities may rise because the 50-year flood is based
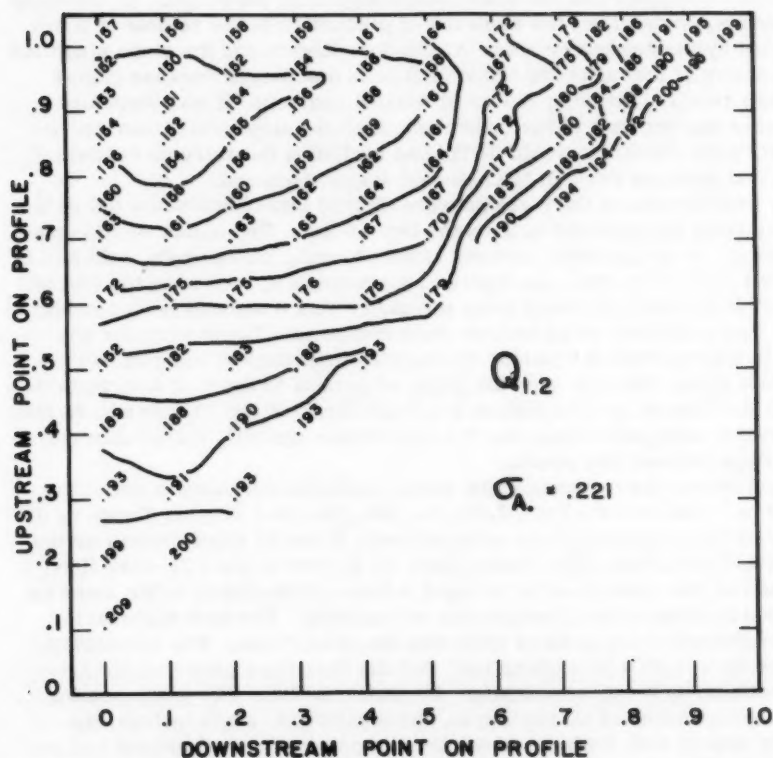
FIG. I   VARIATION OF STANDARD ERROR WITH SLOPE FACTOR

on 40 items, whereas the others are based on 170 items. They may also be due to the less dependable data available for the 50-year floods.

Comparison is shown with the graphical study (using channel slope No. 1) at the same stages. Results by the datatron and graphical method are not strictly comparable, first because of the difference in methods, second because 85 to 95 items were used in the graphical analysis and 170 items in the datatron analysis. However, the final datatron results, using the "85-10" slope, indicate practically the same standard error as the channel slope No. 1 used in the graphical analysis. This is gratifying because the "85-10" slope is far simpler to compute.

Figure 3 shows the typical regression equation with drainage area and slope and a plot of the regression coefficients. The consistent variation in regression coefficients is evident. (The coefficient, b, for the drainage area, might well be 1.00 throughout). The multiple correlation coefficients for each of the regressions range between 0.93 and 0.97.

Opportunity still exists for improvement in the standard errors shown in Table 2, by the use of other parameters employed in the graphical study or by the improvement of parameters such as precipitation indices. Further
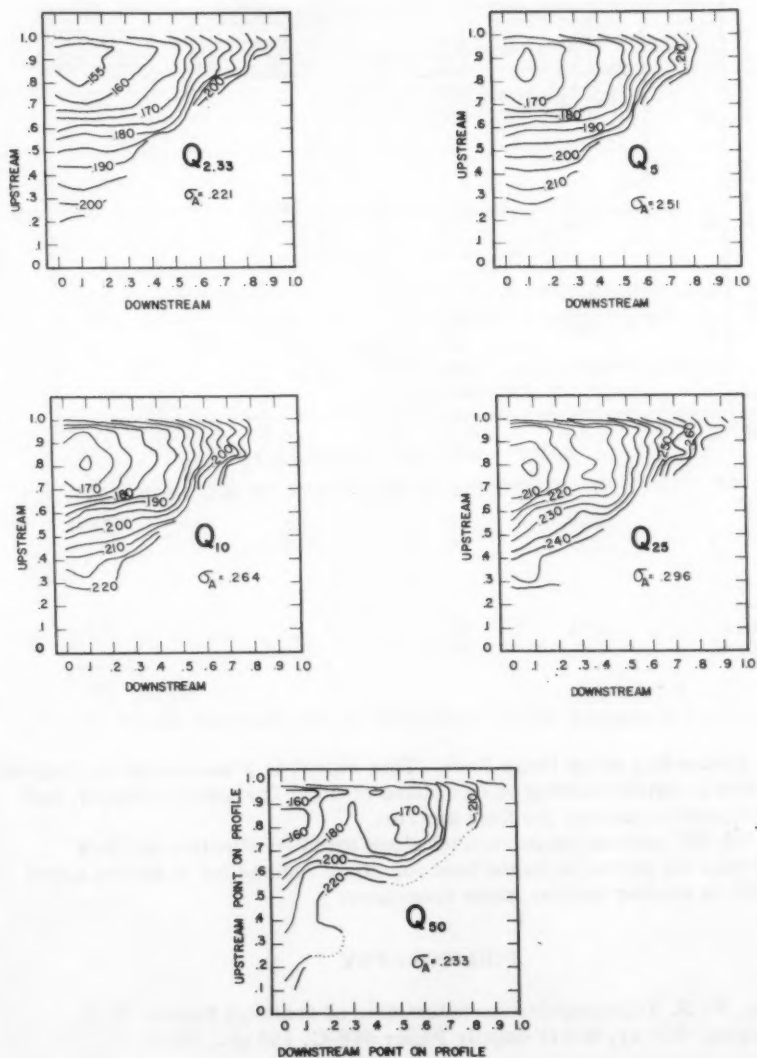
FIGURE 2: VARIATION OF STANDARD ERROR WITH SLOPE FACTOR

Table 2.- Multiple Correlation with Drainage
Area and Slope (85 to 10%)

	Standard error			
	Datatron		Graphical	% Error explained by slope
$Q_{1.2}$	log units	ave. %	ave. %	
A	0.221	53	-	34
A, S	.152	35	-	
$Q_{2.33}$				
A	.221	53	58	32
A, S	.153	36	*36	
Q_5				
A	.251	61	-	38
A, S	.164	38	-	
Q_{10}				
A	.264	65	78	40
A, S	.165	39	*40	
Q_{25}				
A	.296	74	64	34
A, S	.207	49	*45	
Q_{50}				
A	.233	56	-	34
A, S	.158	37	-	
			Ave.	35

*Graphical standard error computed using channel slope No. 1.

work is proceeding along these lines. This report is presented as a progress report with a definite finding on one phase of flood-frequency analysis, and is not intended to provide the final answer.

The "85-10" percent main-channel slope found so effective for New England may not prove to be the best for other regions but it should serve as a guide in similar studies made elsewhere.

BIBLIOGRAPHY

Langbein, W. B. Topographic characteristics of drainage basins, U. S. Geological Survey, Water-Supply Paper 968-C, 155 pp., 1947.

Bigwood, B. L., and Thomas, M. P., A flood-flow formula for Connecticut, U. S. Geological Survey, Circular 365, 16 pp., 1955.

Knox, C. E., and Nordensen, T. J., Average annual runoff and precipitation in the New England-New York area, U. S. geological Survey, Hydrologic Investigations Atlas HA 7.

Wallis, W. A., and Roberts, H. V., Statistics, a new approach, The Free Press, Glencoe, Illinois, 603, 604, 1956.
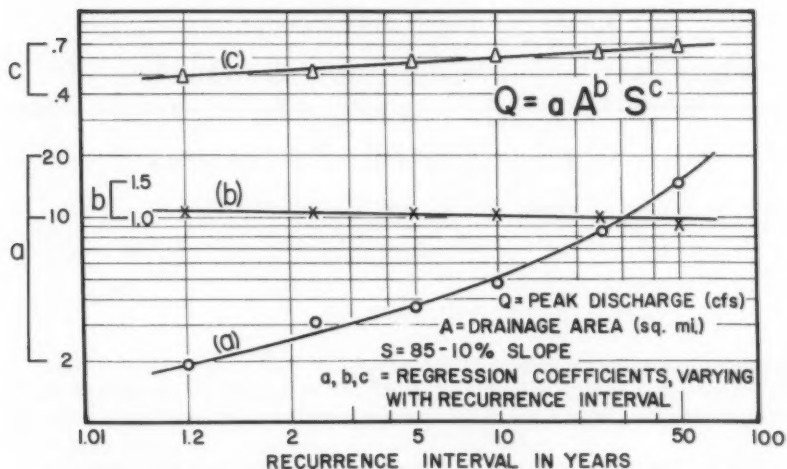
FIG. 3   VARIATION OF REGRESSION COEFFICIENTS WITH SIZE OF FLOOD

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

STORM WATER DRAINAGE IN THE CHICAGO AREA

Horace P. Ramey,[1] M. ASCE

SYNOPSIS

This paper, read before the Hydraulic Division of the American Society of Civil Engineers on February 25, 1958, is a discussion of past and recent flooding conditions in the Chicago area and some recommendations for their improvement.

The opinions and recommendations stated herein are based on fifty years of investigation and experience with these problems. The recommendations discussed are not meant to be specific, but rather to assist future students of the subject in arriving at definite solutions.

Floods have plagued mankind since the great Deluge of 2349 B.C. That was about 4000 years ago. One might think that the Chicago area, much of it on a Continental Divide, with drainage both into Lake Michigan and into the Mississippi Valley, would be comparatively free from floods, but such is not the case.

To protect Lake Michigan, the source of its public water supply, from contamination by sewage, Chicago decided many years ago to keep its drainage out of the lake. This eliminated the quickest outlet for storm drainage, and, because of the topography, placed the main outlet for such drainage at the southwesterly corner of the Chicago area. This, and the generally flat terrain of the region, has produced many problems, some of which are yet to be solved.

In the matter of flood runoff, the Chicago region is served by three streams: the Chicago River, which drains the northern area of the city itself as far south as 87th Street; the Des Plaines River, which drains the westerly areas; and the Calumet River, which drains the entire Calumet region south of 87th Street.

1. Chf. Engr., The Metropolitan Sanitary Dist. of Greater Chicago, Chicago, Ill.

A knowledge of the topography of the Chicago region is essential to the study of its flooding; and a study of the past furnishes the best data for the solution of future problems.

Topography, Drainage Areas

The most important stream is, of course, the Chicago River, which extends due west from Lake Michigan, a little more than a mile, to the Forks. From this point the North Branch extends north by northwest, about twenty-nine miles, to its source in the Skokie Valley, west of Highland Park and Lake Forest, in Lake County, Illinois. The lower six-mile reach of the North Branch of the Chicago River has been improved for navigation and flood flow, to Lawrence Avenue. Beyond that point the North Shore Channel of The Metropolitan Sanitary District of Greater Chicago extends eight miles to a connection with Lake Michigan, at Wilmette. This channel (dry weather flow capacity, 1000 cfs) passes the drainage of the North Shore towns, Evanston to Glencoe, into the North Branch.

About three blocks north of Lawrence Avenue a dam was constructed to hold the North Branch of the Chicago River in its natural state, hydraulically. Above this dam, the North Branch extends westerly and then northerly about eight miles and then divides into three small streams, the West Fork, the North Branch, and Skokie River. The Skokie Marsh covered a five-mile reach of this latter river, west of Winnetka and Glencoe, and, in its natural state, served to retain certain flood water. Much of this marsh has been drained and the area used for residential building; and lagoons have been constructed, to control to some extent the flow of water in the Skokie River.

The South Branch of the Chicago River, extended from the Forks (Lake Street) due south about two miles, and then southwesterly two miles further, to Ashland Avenue and 26th Street, where it divided into the South Fork and the West Fork. The South Fork extended south one and one-half miles to 39th Street and Racine Avenue. The West Fork extended west about two and one-half miles to Kedzie Avenue and 32nd Street, the easterly limit of a slough known as Mud Lake, which was the source of the South Branch. Mud Lake extended southwesterly, about six miles, to Harlem Avenue and 49th Street, where it connected with the Des Plaines River.

The Main Drainage Canal was constructed from the West Fork of the South Branch of the Chicago River, at Damen Avenue (one-half mile west of Ashland Avenue) to extend southwesterly across the Continental Divide just south of Mud Lake, past Summit, and down the Des Plaines Valley, to connect with the Des Plaines River at Lockport, Illinois.

Present drainage areas of the Chicago River (above Summit) are 309 square miles.

The Des Plaines River, rising in southeastern Wisconsin, west of Racine, flows southward, parallel to and about ten miles west of the west shore of Lake Michigan, to about 49th Street and Harlem Avenue, Chicago (one mile south of Riverside), and then makes a sharp bend to the southwest and continues past Lockport and Joliet, to Dresden, where it unites with the Kankakee River to form the Illinois River. Drainage area of the Des Plaines River, above Riverside, is 643 square miles; above Lemont, 690 square miles; and above Lockport, 713 square miles.

The most important tributary of the Des Plaines River is Salt Creek, which comes in from the west at about 39th Street. Salt Creek rises in Cook County, northeast of Palatine, flows southward through the eastern section of DuPage County, past Bensenville, Villa Park, and Elmhurst, to Hinsdale; and then flows eastward past Western Springs and LaGrange Park, through Bookfield, into the Des Plaines River. Its drainage area is about 152 square miles.

Flag Creek rises in the southern part of Hinsdale and Western Springs and flows south to about 87th Street, then southwesterly and into the Des Plaines River about half-way between Willow Springs and Sag. Its drainage area is about 20 square miles.

The Little Calumet River rises in Laporte, Indiana, and flows westward into Illinois at about 170th Street, then northwesterly to a point in Blue Island about one-half mile west of Ashland Avenue, at 136th Street. Then it bends sharply to the east and flows about seven miles to a junction with the Grand Calumet River, at Torrence Avenue and 139th Street. From this junction the Calumet River (drainage area 38 square miles) extends north about six miles, past Lake Calumet, to Lake Michigan at 90th Street. From this same junction, the Grand Calumet River (drainage area 50.1 sq. mi.) extends eastward to the Lake Michigan shore near the east line of Gary, Ind. Only a sand bar separates this river from the lake. The Grand Calumet is really a bayou, having once been the outlet channel of the Little Calumet.

The Indiana Harbor Ship Canal connects the Grand Calumet River with Lake Michigan, through East Chicago, Indiana. The drainage area of the Grand Calumet, east of this canal, is 30.8 square miles; and west of it, 19.3 square miles.

The original watershed of the Little Calumet River, above Riverdale, was about 670 square miles. This included about 75 square miles in the eastern half of the Sag Valley, drained by Stony Creek, a tributary of the Little Calumet, and seven square miles in the Blue Island area. It also included about 370 square miles, in Indiana, in the upper reaches of the Little Calumet, which, since 1925 have drained directly into Lake Michigan, near Dune Park, Indiana, through Burns Ditch. Deducting these areas, total 452 square miles, from the original 670 square miles, leaves the present watershed of the Little Calumet, above Blue Island (its junction with the Calumet-Sag Channel) as approximately 218 square miles.

The drainage area of the Little Calumet River, between Blue Island and its junction with the Grand Calumet, is eight square miles, plus thirteen square miles tributary to the Calumet storm water pumping station; total thirty-one square miles.

The present drainage area of the Little Calumet River, as reversed, to flow into the Calumet-Sag Channel, at Blue Island, is 306 square miles.

The drainage area of the Calumet-Sag Channel, east of the Continental Divide, is about eighty-two square miles; and west of this Divide, about twelve square miles.

The most important tributary of the Little Calumet River is Thorn Creek (drainage area 115 square miles), which rises southwest of Chicago Heights and flows into the Little Calumet in the western part of Lansing, Ill. One of the principal tributaries of Thorn Creek is Butterfield Creek (drainage area about thirty square miles), which rises in the southwest corner of Cook County, west of Matteson, and flows through Olympia Fields, and then south of Flossmoor and Homewood, to join Thorn Creek in Glenwood. An easterly tributary of Thorn Creek is Deer Creek (drainage area about sixteen square

miles), which rises in Crete and flows northerly through East Chicago
Heights, and then northwest along the Glenwood-Dyer Road, into Thorn Creek
in Glenwood.

The area south of 159th Street and west of the Illinois Central Railroad is
drained by the Calumet Union Drainage Ditch (drainage area about twenty-four
square miles), which runs easterly from Kedzie Avenue, along the line of
163rd Street, and into the Little Calumet River in South Holland.

Summary—General Topographic Characteristics

The area discussed herein may generally be described as flat and rolling
and although there are many ridges and divides in the area, they are not obvi-
ously distinct.

The existence of a large flood plain is evident in those areas containing
ponds and sloughs as well as the areas bordering many of the shallow stream
beds.

Most of the streams in the area flow parallel to the lake shore as a result
of the terminal moraines created by the receding glaciers.

Changes in Stream Conditions

The first change from the natural conditions of these streams, in regard
to storm flow, occurred in 1848, when the Illinois and Michigan Canal was
opened, from the South Branch of the Chicago River at Bridgeport (Ashland
Avenue) through the Des Plaines Valley, to Lockport. About a year later, the
Calumet Feeder of the I. & M. Canal went into service, from the Calumet
River near Blue Island, through the Calumet-Sag Valley to the I. & M. Canal,
at Sag.

Between 1867 and 1871, the Illinois and Michigan Canal was deepened, from
Bridgeport to Lockport, for a gravity flow of 1,000 cfs from the Chicago
River, in lieu of about 250 cfs previously pumped.

In 1871, the Ogden-Wentworth Ditch was dug, from the Des Plaines River,
through Mud Lake, to the West Fork of the South Branch of the Chicago River.
The purpose was to drain Mud Lake, to enhance local real estate values. This
ditch was 20 feet wide and its bottom was below the bed of the Des Plaines
River. In the spring of 1872, the Des Plaines River floods greatly enlarged
the new ditch, depositing the soil in the I. & M. Canal.

Thereafter, more flood water than normal came from the Des Plaines
River into the West Fork of the South Branch. Even the dry weather flow of
the Des Plaines River followed the same course and then flowed down the I. &
M. Canal. This condition continued until June 1877, when the Ogden Dam was
built.

The Ogden Dam was a row of sheet piling, 50 feet long, with crest at ele-
vation +11.8 CCD, which was about two feet lower than the natural banks of
the Des Plaines River, and a foot or two higher than the surface of Mud Lake.
It was built in the arm of the Des Plaines River which led to Mud Lake, west
of Harlem Avenue, near 49th Street. This dam was rebuilt in 1885, of stone-
filled timber cribs, with its crest at the same elevation. The new dam served
until 1894, when the Riverside Spillway, 397 feet long, at elevation +16.25
CCD, was built on the east side of the Des Plaines, a short distance north of
Ogden Dam.

MAP OF
CHICAGO
AND VICINITY
Scale

LAKE MICHIGAN

CHICAGO RIVER BASIN

SALT CREEK

DES PLAINES RIVER BASIN

CALUMET BASIN

DRAINAGE AREAS
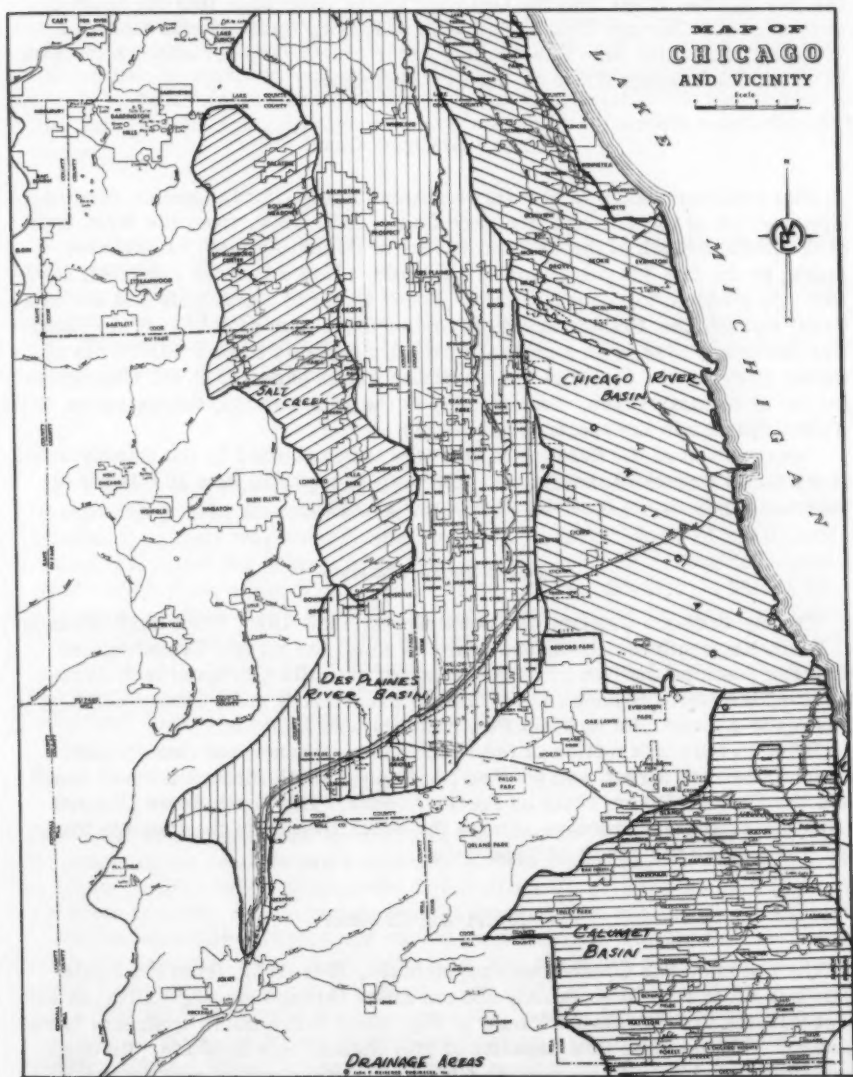
The Riverside Spillway was closed in 1908 when the Willow Springs Spillway was built, to replace it, and discharge a like amount of flood water from the Des Plaines River into the Drainage Canal, instead of into the South Branch of the Chicago River. The Willow Springs Spillway was closed in 1955, and since that date Des Plaines River floods have not affected the flow in either the Chicago River or the Drainage Canal.

Chicago Drainage Canal

The greatest change in stream conditions in the Chicago region was the construction of the Chicago Drainage Canal, 1892-1900, from the West Fork of the South Branch of the Chicago River, at Damen Avenue, twenty-nine miles, to the Des Plaines River at Lockport. This was later extended, 1903-1907, four miles, to connect with the Upper Basin of the Illinois and Michigan Canal and the Des Plaines River, in the northern part of Joliet. This channel was designed, 1892, with a normal hydraulic capacity to flow 10,000 cfs of water, to carry the runoff of the maximum storm expected in the Chicago area for the next thirty years. Its capacity, in the rock section, fifteen miles, from Willow Springs to Lockport, is 14,000 cfs.

Construction of the Drainage Canal was accomplished by the improvement of the South Branch of the Chicago River, to capacity to flow 10,000 cfs of water at a velocity of less than 3 feet per second.

North Shore Channel

The North Shore Channel was constructed, 1908-1910, from Lake Michigan, at Wilmette, southwest about two miles to Emerson Street, Evanston, and then due south on the line of Kedzie Avenue, six miles, to Lawrence Avenue. It joined the North Branch of the Chicago River about three blocks north of Lawrence Avenue. Its nominal flow capacity was 1000 cfs.

Prior to the construction of the North Shore Channel, the marshy land around Emerson Street, and southward, drained south through a small brook into the Chicago River, south of Foster Avenue. The North Shore Channel added no significant drainage area to the North Branch of the Chicago River, but it did speed the runoff of flood water.

Calumet-Sag Channel

The Calumet-Sag Channel was constructed, 1911-1922, from the Little Calumet River near Blue Island, sixteen miles through the Sag Valley, to connect with the Main Drainage Canal, at Sag, about three miles upstream from Lemont. The nominal flow capacity of this channel was 2000 cfs. Its depth was 20 feet and its minimum width 60 feet. This channel is now being widened, by the Federal Government, for navigation, to a bottom width of 225 feet, at a depth of 9 feet below water. This will increase its nominal hydraulic capacity to 5000 cfs.

Summary Regarding Man Made Changes—General Effects

The areas discussed in this paper are densely populated and highly industrialized. Some of the stream characteristics have been altered by dredging and the addition of many man made structures.

An effect of these man made changes has been to change, in some cases, the outlying borders of the drainage basins. This, together with the general topographic characteristics of gently sloping plains and flattish slopes, presents a drainage problem which requires bold and imaginative solution.

Considerable progress has been made in the past with the creation of the existing open channel system, as well as the sewer program presently under way. It is, however, obvious that the tremendous population explosion and rapid rate of urbanization require immediate action to remedy a deteriorating situation.

Introduction to Flood History

Considerable material describing early floods in the Chicago region is available in the form of historic rhetoric.

Some rainfall data as far back as 1875 is available for use. This data is, however, not sufficiently complete to afford any scientific analysis of their effect. The following briefly summarizes the general impressions of those recorded floods and periods of intense rainfall.

Early Floods

The first record of a flood in the Chicago region was March 29, 1674, by the explorer priest, Marquette. He and two other Frenchmen and a number of Indians, on their way from Green Bay to the principal town of the Illinois Indians, had spent the winter, after December 4, 1673, near Damen Avenue and the West Fork of the South Branch of the Chicago River. On March 29, 1674, he was driven from his camp by high water coming through Mud Lake, from the Des Plaines River, with the spring break-up of the ice. He proceeded, the next day, up the South Branch, through Mud Lake, without any portage, and then on down the Des Plaines River, which was in flood (Brown's "Drainage Channel and Waterway," pages 103-104).

Flood of March 12, 1849

A most disastrous flood occurred in the Chicago River, March 12, 1849. No precipitation records are available, but this flood followed the pattern of previous floods coming in from the Des Plaines River, through Mud Lake, into the South Branch. An ice jam formed in the South Branch, south (upstream) of the Madison Street bridge and raised the water and floating ice two to three feet. The jam broke at 9 A.M., March 12, and the rapid current swept away every vessel moored below that point. The mass of ice and sailing vessels struck the Madison Street bridge, wrecked it, and added the bridge to the floating debris. Next, the Randolph Street bridge was swept away, as were a number of canal boats and sailing vessels moored in the South Branch, north of Randolph Street.

The mass of ruin then swept into the main river, where the banks were lined with steamers, schooners, brigs, canal boats, scows, etc. The Wells Street bridge was open and the middle of the stream clear, but the floating mass of vessels and wreckage extended from bank to bank and soon swept away both ends of this bridge and mingled it with the general wreckage. The larger vessels crashed together, entangling booms, bowsprits, and rigging.

LOCATION MAP

PRECIPITATION STATIONS FURNISHING DATA

MAIN CHANNEL PROFILE

MAIN CHANNEL PROFILE

Blue Island Locks

Normal Water Surface

Harlem Ave.

South Wall Main Channel

Bottom Profile   .000027

FEET

CAL SAG CHANNEL
PROFILE

MILES

NORTH SHORE CHANNEL

MILES
DES PLAINES RIVER
HOFFMAN DAM TO LAKE & COOK CNTY LINE

NORMAL WATER SURFACE

BOTTOM PROFILE

MILES
DES PLAINES RIVER
HOFFMAN DAM TO LOCKPORT

FEET

LOCKPORT RD.

ROMEO RD.

FLAG CREEK

WILLOW SPRINGS SPILLWAY

SUMMIT ROAD

OGDEN AVE.

HOFFMAN DAM

HY 4

The s
feet a

Th
the la
Stree
Above
below
were

Th
passe
crisi
add t

Th
and I

O
rains

H
flood

Sum

F
Chic

The smaller vessels were crushed. The ice, in places, was wedged several feet above water level.

The Clark Street bridge was completely swept away and a great many of the larger vessels became wedged together a short distance east of Clark Street, and with the general wreckage formed a dam, by 12 o'clock noon. Above this dam, ice and wreckage accumulated to considerable height; and below it, numbers of vessels were swept into Lake Michigan, many of which were wrecked.

The dam still held at 10:00 A.M. the following day, but most of the ice had passed under the vessels and the water level had fallen about two feet. The crisis was over. Fears that the ice in the North Branch would break up and add to the damage, were not realized.

The flood was general and caused considerable damage in the Des Plaines and Illinois Valleys.

Other Floods

Other floods occurred in Chicago, due to the following extremely heavy rains:

	Rainfall
September 9, 1875	3.44 inches
July 25-26, 1878	4.14 inches
July 6-7, 1879	3.25 inches
November 11-12, 1881	3.35 inches
November 5-6, 1883	3.39 inches
March 25-26, 1884	3.26 inches
June 2-3, 1885	3.44 inches

However, the records available are not sufficient for any study of these floods.

Summary of Runoff from Past Floods

For convenience in study, the principal data regarding past floods in the Chicago area are listed in the tabulation following:

	Rainfall Inches	Time Hours	Runoff cfs
August 2-3, 1885	6.33	27	?
May 6-8, 1892	?	?	10,000
August 11, 1908	4.34	24	9,000
April 29, 1909	2.92	36	8,900
August 14-15, 1909	3.52	24	10,000
March 17-18, 1919	2.31	48	10,600
May 3-6, 1919	2.77	72	9,000
Sept. 11-13, 1936	4.58	39	15,000
July 1, 1938	4.14	16	12,000
July 6, 1943	3.9-4.54	24	16,000

April 5-7, 1947	5.27	24	29,100
March 19, 1948	3.50	13	24,500
July 14-15, 1949	4.64	24	16,000
June 2, 1950	4.38	24	13,000
July 16-17, 1950	5.40	24	16,000
May 10-11, 1951	3.91	24	16,000
October 9-11, 1954	6.66	28	27,000
	7.20	47	
July 12-13, 1957	6.00	6	28,000
	6.34	23	

The conclusion to be drawn from a study of these floods is that in the Chicago area within the past thirty years the runoff, from one cause or another, has increased at least two and one-half times. The Chicago Drainage Canal, planned in 1892 to carry the runoff from the greatest flood which might occur in the Chicago area, then estimated at 10,000 cfs, has now become inadequate. Many changes have taken place in the Chicago area since that time, sixty-five years ago, and the whole storm drainage problem must be re-evaluated. Apparently outlet capacity of 30,000 to 35,000 cfs will be needed.

Floods in the Des Plaines River

The Des Plaines River has a drainage area of 643 square miles, above Riverside. Of this, some 152 square miles are in the watershed of its principal tributary, Salt Creek. The drainage area of Salt Creek, above the Fullersburg dam, just north of Hinsdale, is about 122 square miles.

From the Hoffman dam, in Riverside, the Des Plaines River extends thirty-one miles north to the Cook-Lake County line, just north of Wheeling, and on fifty-seven miles further to its source in Wisconsin, west of Kenosha and Racine. Downstream from Riverside this river extends southwesterly, about twenty-six miles, to Lockport. The slope in the flood plain of the river, and in the river itself, averages about 1.0 feet per mile between Riverside and the north line of Cook County, and about 1.9 feet per mile between Lockport and Riverside.

The average cross section of the Des Plaines River, between Riverside and North Avenue, for dry weather flow, is slightly less than 400 square feet; and the section between North Avenue and Wheeling is probably less. This means a channel about 150 feet wide and less than 3 feet deep.

The Forest Preserve District of Cook County, in conjunction with the State of Illinois, has adopted a project to control the low water flow of the Des Plaines River, which at times is as low as 5 cfs, for weeks on end, in the thirty-one mile stretch between Riverside and the Cook-Lake County line. This involved the construction of twelve to thirteen low head dams, to provide at least two feet of water above each dam. Dams at Dempster Street, Devon Avenue, and Touhy Avenue were built in 1946. The Hoffman dam, in Riverside, was rebuilt in 1950-1956. Dams at Hintz Road, one mile south of Wheeling, and at Foundry Road, were to replace existing dams. One other dam was built in 1957.

At least twenty-nine highway and nine railroad bridges cross this stretch of the Des Plaines River, the average horizontal clearance of such bridges being about 150 feet. The average loss of head through these bridge openings in times of high water, may be 3 to 4 inches per bridge. The flood stage at North Avenue may therefore be 3 to 4 feet higher because of losses through these restricted openings. The flood stage at Wheeling may be increased twice as much. Accumulated drift at bridges and other narrow places has, no doubt, caused further losses, for there is no record of this river having been cleared during the past twenty-five years.

Some recent floods in the Des Plaines River, as measured at Riverside, are as follows:

	Riverside Gauge	Maximum Discharge cfs	Runoff cfs per sq. mi.
April 21, 1881	-	13,500	21.0
February 9, 1887	20.2	10,324	16.1
June 24, 1892	21.3	13,000	20.2
March 25, 1904	21.45	10,780	16.8
March 7, 1908	21.47	10,810	16.8
May 1, 1909	21.97	11,620	18.1
March 18, 1919	22.17	7,450 ?	11.6
May 9, 1933		5,200	8.1
Sept. 17, 1936		5,160	8.1
July 3, 1938		5,350	8.3
March 16, 1944		4,600	7.2
April 6, 1947		3,780	5.9
March 20, 1948		6,510	10.1
April 26, 1950		6,340	9.9
October 11, 1954		6,340	9.9

The measurement, April 21, 1881, was made by Lyman E. Cooley, of flow over a weir. The measurement, February 9, 1887, was made by T. T. Johnson, using rod floats.

The flood in April 1881 was due to the melting of ice and snow, amounting to about one foot of water, which had accumulated on ground which was practically saturated when winter set in.

Four floods in Salt Creek, as measured at the Fullersburg dam, in 1876, 1881, 1886, and 1887 produced flows of about 2800 cfs. In these cases, the runoff was about 22.6 cfs per square mile, from the 122 square miles in the watershed above Fullersburg. The actual measurement in 1881 was 2760 cfs.

The floods of 1881, 1892, and perhaps of 1909 may be classed as great floods. It is fortunate for the residents of the Des Plaines Valley that no great flood has occurred in the Des Plaines River since 1909. The highest water levels in this time were in March 1919; and these levels were about

3.5 feet below the levels due to the 1881 flood. The recorded discharge at Riverside, March 18, 1919, was 7450 cfs. However, with a discharge of 6510 cfs March 20, 1948 and 6340 cfs April 26, 1950, the water levels in the Des Plaines River between Madison Street and North Avenue closely approached the high levels of March 1919. This indicates some obstruction to flow and the need for clearing the channel.

A report on the "Improvements of the Des Plaines River" was made July 1, 1924, by Ernest L. Cooley, Hydraulic Engineer, to Edward J. Kelly, Chief Engineer of The Sanitary District of Chicago. For the critical reach between the Hoffman dam, at Riverside, and North Avenue, the improvement recommended was as follows:

> Lower the crest of the Hoffman dam from +25.56 to elevation +20.00 C.C.D.;
> Excavate the channel from the Hoffman dam, 4800 feet to the junction with Salt Creek, to a bottom width of 300 feet and a bottom grade of +15.00;
> Excavate the channel, from the Salt Creek junction to a point about a mile north of North Avenue (Elmwood Park), to a bottom width of 200 feet and a bottom grade rising to +17.00 at 22nd Street, +20.00 at Lake Street, and +22.00 at North Avenue;
> Construct a sub-dam at 22nd Street with crest at elevation +22.00;
> Lower the crest of the Public Service dam, below Lake Street, from +28.54 to elevation +25.00.

This plan, estimated to cost several millions of dollars, would lower the flood water stage by 5 or 6 feet, from the Riverside dam to North Avenue and beyond. The dams would cause low water boating pools, 18,000 feet long through Riverside to 22nd Street, 16,000 feet long from 22nd Street to Lake Street, and 15,000 feet long from Lake Street to Elmwood Park.

This was a good plan in 1924 and it is a good plan today, with proper modifications, including its extension to the north line of Cook County. Recent studies indicate that the bottom grade of the channel should be about +13.00 C.C.D. from the Hoffman dam to the mouth of Salt Creek and then rise, on a slope of 0.002, to about +17.20 at 12th Street, +21.20 at North Avenue, +33.00 at Golf Road, +36.00 at Lake-Euclid Avenue, and +45.00 at the north line of Cook County. The depth, below high water, would be 10 feet. Bottom width of the new channel would be 300 feet, from the Hoffman dam to the mouth of Salt Creek, then about 230 feet to North Avenue, then 210 feet to Higgins Road, diminishing to 150 feet at Palatine Road. Changes in channel widths would be made at points where tributary creeks enter the Des Plaines River. To maintain desired low water levels, sub-dams should be constructed, at proper intervals. To lessen the rise in water level over these dams, to pass high flood flows, the dams could be built diagonally instead of directly across the stream, and thus be given greater crest lengths. Growth of shrubbery, which would be obstructive to flood flow, should be restricted.

It is believed that channel improvements, such as those suggested above, would enable the Des Plaines River to carry any flood which may reasonably be expected in the future. The depth of cut, below the bed of the present channel, would probably average 4 to 5 feet, between the Hoffman dam and North Avenue, and perhaps 3 feet between North Avenue and the County Line. The construction cost might be $6,000,000 to $9,000,000.

Improvement would also be needed in Salt Creek, Addison Creek, Silver Creek, and other tributaries. Reports have been made, by the Division of Waterways of the State of Illinois, indicating that such construction might cost, as follows:

Addison and Silver Creeks	$4,790,520
Flag Creek	1,836,170
Salt Creek	5,000,000

Flag Creek (drainage area about 20 square miles), which rises in the southern part of Hinsdale and Western Springs, north of 47th Street, is a small stream, about seven miles long. The Illinois Toll Road (now under construction) parallels Flag Creek, from 71st Street, north to about 47th Street, and crosses it. Considerable flooding has occurred, in the past, in Hinsdale and Western Springs, because of the inadequate natural channel of Flag Creek.

In recent years a considerable amount of building has been permitted on low lands subject to overflow from high floods in the Des Plaines River. Innocent purchasers of houses in such areas have suffered unexpected and unwarranted damage from high water. No public agency should approve such construction, regardless of the pressure applied by subdividers and builders.

Little Calumet River

A study of floods in the Little Calumet River indicates that storm runoff rates in the Calumet area have at least doubled in the past fifty years. The drainage area of this stream has been subject to several changes, over the years. The original 670 square miles of the Little Calumet above Riverdale, included seventy-five square miles in the Sag Valley and seven square miles around Blue Island, now tributary to the Calumet-Sag Channel, as well as 370 square miles in Indiana which now drain to Lake Michigan through Burns Ditch.

Records of the worst floods are as follows:

	Runoff	Drainage Area Sq. Mi.	Rate per Sq. Mi.
Riverdale			
March 6-7, 1908	11,000 cfs	670	16.4 cfs
North Harvey			
June 11, 1939	3,440 cfs	198	17.4 cfs
March 16, 1944	4,305 cfs	198	21.7 cfs
Blue Island			
April 5-7, 1947	9,100 cfs	306	29.8 cfs
March 19, 1948	5,000 cfs	306	16.3 cfs
October 9-11, 1954	6,700 cfs	306	21.9 cfs
July 12-13, 1957	7,200 cfs	306	23.5 cfs

A new lock and control gates are to be built in the Calumet River as part of the project for widening the Calumet-Sag Channel, now under construction. This will leave only the thirty-eight square miles of the Calumet River watershed tributary to Lake Michigan. The remaining 268 miles of the present 306

square miles of watershed of the Little Calumet, Grand Calumet, and Calumet, will be tributary to the Calumet-Sag Canal.

For present flood relief into Lake Michigan, a channel 3,500 to 4,000 square feet in cross-section is available. In the future, only the capacity of the control gates at the new lock will be available.

It would appear that the minimum capacity of the Calumet-Sag Channel, for the passage of the runoff from a great flood, should be about 10,000 cfs. This much, or more, could come from a runoff of 30 cfs per square mile, from the 306 square mile watershed east of Blue Island and the 96 square miles in the Calumet-Sag valley. About 20 per cent of the 221 square miles within the city limits of Chicago, or 44 square miles, south of 87th Street, are included in the area draining to the Little Calumet River and the Calumet-Sag Channel.

The channel of the Little Calumet River, upstream from Blue Island, is quite inadequate for storm water flow. In 1941 the State of Illinois improved this channel for a 50-foot width, from the Indiana State line to the mouth of Thorn Creek; and in 1946 extended this 50-foot improvement to Roll Street, in Blue Island. Under present conditions of greatly increased runoff due to the considerable building in the area, it appears that the width of this channel should be more than doubled and perhaps the depth should be increased. Openings at the many bridges should be greatly increased. A pending improvement in Thorn Creek, already authorized by the State of Illinois, will increase the rate of storm water flow; and direct attention to the need for further improvement in the Little Calumet River above Blue Island.

Summary—Floods: Need for Remedial Measures Obvious

The foregoing has outlined chronologically, the history of storms and resultant floods in the Chicago region. As can be seen, there exists some data of recent years which can be used to appraise and investigate a few past storms and their relation to flooding. It becomes more obvious upon such examination that immediate remedies must be found for the flooding situation in the area. The flood frequency is becoming greater with successively smaller rainfall and promises to become still greater with further urbanization and growth.

City of Chicago

The City of Chicago embarked on a program of construction of storm relief sewers, in 1947, expected to entail an ultimate expenditure of $165,000,000, to catch up with existing needs. Three bond issues to provide funds for this construction have already been approved by the voters, as follows:

1946	$58,160,000
1955	30,000,000
1957	8,000,000
Total approved	$96,160,000
Future	70,000,000
Total	$166,160,000

It is hoped that this work can be carried on at the rate of about $15,000,000 per year, 1957 and after.

In addition to the work by the City of Chicago, The Sanitary District of Chicago expended $47,367,000 on its system of South Side relief sewers (to December 31, 1956); and is currently spending about $5,200,000 on an extension of the Racine Avenue Pumping Station, for pumping storm water only. The total expenditure, by the Sanitary District, for this storm drainage will be about $52,570,000.

This system of sewers with the existing 39th Street Conduit was designed to carry 5500 cfs of storm runoff, from 27.6 square miles, between 31st Street and 87th Street, and east of Racine Avenue to Lake Michigan; or at a runoff rate of 200 cfs per square mile. These sewers have capacity to carry 400 cfs of sewage, in addition to the 5500 cfs of storm flow, total 5900 cfs.

The sewer outlet capacity of the City of Chicago, past and prospective, including the South Side sewer system of the Sanitary District, is indicated in the tabulation following. Runoff rates are based on the present 221 square miles of area within the City of Chicago.

Years	Sewer Outlet Capacity	Runoff cfs per sq. mi.
1900-1910	10,000 cfs	45.5
1920	14,000 cfs	64.0
1930	20,000 cfs	91.0
1940	30,000 cfs	137.0
1950	35,000 cfs	159.0
1957	37,000 cfs	173.0
1960	41,000 cfs	187.0
1970	55,000 cfs	250.0

Sewer outlet capacity, thirty years ago, was about ten times the dry weather flow. The outlet sewers now being constructed are designed with capacities thirty to fifty times the dry weather flow. This will be reduced by high water levels in the Chicago River and the Drainage Canals; but the storage capacities in these large relief sewers will be considerable.

These present and prospective sewers are designed to flow their full capacities into the rivers or canals at a water level of +1.50 CCD. This water level may reach +4.00 in the Chicago and Calumet Rivers and the effect of this would be to reduce the flow capacities of these sewers to 70 or 75 per cent of their designed capacities; or to reduce the total (1970) capacity from 55,000 cfs to about 40,000 cfs.

It is not expected that the flow capacities of all these outlet sewers could ever be developed at the same time. The times of concentrations of floods in the various sewers and sewer systems range from less than thirty minutes, up to three hours. However, a 6-inch rain in six hours, such as fell in the entire Chicago area on July 12, 1957, could, through this completed sewer system, deliver an amazing amount of storm runoff into the Chicago and Calumet Rivers and the connecting canals. In July 1957 a lot of this went into the basement of householders, because of the inadequacy of the local sewers.

No doubt these sewers, in particular, the recent auxiliary outlet sewers, have been designed on a liberal basis. However, if they have not been greatly

over-designed, a study of how much flood water they can deliver to the outlet streams may be a more realistic method of estimating the volume of a future great flood, than the method outlined on previous pages, of estimating it from the amounts of water which have been discharged through the Drainage Canal, in times of past floods.

A study of these outlet sewers, in only the 221 square miles within the city limits of Chicago, indicates the following:

	Percentage of Sewer Capacity	Sewer Capacity - cfs		
		Total	75%	50%
North of Lawrence Avenue	10%	5,500	4,120	2,750
Lawrence Avenue to Lake Street	17%	9,350	7,010	4,670
Total North Branch	27%	14,850	11,130	7,420
South Branch Chicago River	30%	16,500	12,370	8,250
Total Chicago River	57%	31,350	23,500	15,670
Drainage Canal, Western Ave. to Harlem Ave.	16%	8,800	6,600	4,400
Total to Main Channel	73%	40,150	30,100	20,070
Calumet Area, South of 87th Street	27%	14,850	11,130	7,430
Total, City of Chicago	100%	55,000	41,230	27,500

Storm Flow—Computed from Sewer Capacity

It may be assumed, as quite probable, that at least 50 per cent of the total flow capacity of these sewers will be developed simultaneously, in some future flood, and that the storm runoff from the 221 square miles within the present Chicago city limits will be at least 27,500 cfs. To this must be added the storm runoff from about 100 square miles, north of Chicago, in the upper reaches of the North Branch and of the North Shore Channe, including Evanston and the other North Shore towns; and the runoff from about 260 square miles tributary to the Little Calumet River, south of the city. Such runoff may reasonably be calculated at 30 cfs per square mile; and would amount to about 3000 cfs on the North Side, and 7800 cfs in the Calumet region.

The 3000 cfs on the North Side, added to the 20,070 cfs total flow to the Main Channel, from the table above, would indicate a storm flow of at least 23,070 cfs in the Main Drainage Canal, above Summit. This runoff, from the 308.9 square miles of drainage area above Summit, would be at an average rate of about 75 cfs per square mile.

The 7800 cfs in the Calumet region, added to the 7430 cfs flow to the Calumet-Sage Channel, from the table above, would indicate a storm flow of about 15,230 cfs in the Calumet-Sag Channel. This runoff, from the 306 square miles tributary to the Calumet-Sag Channel, would be at an average rate of about 50 cfs per square mile. Storm flow from the ninety-six square miles in the Sag Valley is ignored in this calculation, since much of this could have already passed downstream before the peak concentration in the entire Calumet region.

The total storm flow, under this method of calculation, would be 23,070 cfs from the area north of 87th Street plus 15,230 cfs from the Calumet region,

or 38,300 cfs. This can be compared with the conclusion of 30,000 to 35,000 cfs derived (page 25) from a study of the runoff from past floods.

Introduction to Recommendations

The following discussions and tabulation comprise a program of action geared to increase the outlet capacities for the expected flood waters in the years to come. They are by no means the total solution to the problem but only make available sufficient outlet capacities for the rapidly expanding sewer programs. It should be understood that these recommendations are based on the data and experience taken from the operation of the open channel system and are not meant to solve localized problems due to inadequate sewer capacities. This program is suggested as that part of the total program which will fulfill the outlet capacity needs for flood waters for the next fifty years.

Main Outlet Channels

Since storm runoff in the Chicago and Calumet areas has approached 30,000 cfs on three different occasions in the past ten years (March, 1948, October 1954, July 1957), and since building and paving are increasing at very substantial rates, it is apparent that the runoff from a great storm may be 30,000 to 35,000 cfs, in the not distant future. It is obvious that outlet capacity for such storm flow should be at least 30,000 cfs.

Chicago is the one city, on the Great Lakes, in the fortunate position of not having to drink its own sewage, or pass it on for others to drink. However dilute it might be, the thought is repugnant. For the past thirty-five years, or since August 1922, when the Calumet-Sag was opened, Chicago has kept all of its sewage out of Lake Michigan, the source of municipal water supply. It has not been possible to keep the peaks of flood flows from the Calumet Rivers out of the lake, whenever such flood flows have exceeded the hydraulic capacity of the Calumet-Sag Channel. The same is true of the North Shore Channel, where storm runoff in recent years has had a great increase, and where the consequent water level has, on many occasions, been high enough to cause flow over the restraining walls at Wilmette. On only two occasions (October 1954 and July 1957) has it been necessary to permit the Chicago River to flow into Lake Michigan, to prevent damage to Loop property, from high water. These were on occasions of what might be called great storms.

It may be that this policy of Chicago, cherished over the years, of keeping its drainage out of its drinking water, may have to be abandoned; at least during great storms. It is recommended, however, that sufficient outlet capacity may be provided for storm flows so that relief discharge into Lake Michigan can be limited to the peak runoff from great storms.

To that end, a hydraulic study has been made of the present outlet channels and of how much their flow capacities should be increased, to meet the needs in the near future, of discharge capacity of 30,000 cfs, assuming that any flood flow in excess of that amount could be spilled into Lake Michigan. This study, which should be considered as preliminary only, indicates a need for the following improvements:

Main Outlet, Drainage Canal, from Sag to Lockport—
    Widen present channel, from 160 feet to 300 feet, or more.
    This channel would carry about 30,000 cfs, with water surface -3.0
    at Sag and -8.0 at Lockport.

<u>Calumet Outlet</u>, Calumet-Sag Channel, from Blue Island to Sag—
  Present widening to 225 feet, by Government. This channel should
  carry about 10,000 cfs, with water surface -3.0 at Sag and +4.0 in
  Little Calumet River.

<u>Chicago Outlet</u>
  Drainage Canal, from Willow Springs to Sag—
    Widen present channel, from 160 feet to 230 feet. This channel
    should carry about 20,000 cfs, with water surface -1.5 at Willow
    Springs and -3.0 at Sag.

  Drainage Canal, from Summit to Willow Springs—
    No improvement indicated. Present channel should carry about
    20,000 cfs, with water surface +0.1 at Summit and -1.5 at Willow
    Springs.

  Drainage Canal, From Western Aveiiue to Summit—
    Widen present channel 70 feet. The channel should then carry
    about 20,000 cfs, with water surface +1.5 at Western Avenue and
    +0.1 at Summit. Perhaps this reach should be improved to the
    standard dock section, with 240 ft. width between docks; and
    proper deepenings.

<u>Chicago River</u> and Drainage Canal from Lake Street to Western Avenue—
  No improvement indicated. Present channel should carry about
  20,000 cfs, with water surface +4.0 at Lake Street and +1.5 at
  Western Avenue.

These estimates are based on a maximum stage of +4.0 CCD in the Chicago
and Little Calumet Rivers, of -3.0 at Sag, and a minimum of -8.00 at Lock-
port. Holding the water level at Sag at about -3.00 is very important. In the
March 1948 flood, this level was about +1.60. In the October 1954 flood, it
was probably much higher, but the gauge failed. In the July 1957 flood, it was
+0.40. These high water levels at Sag cause an unreasonable decrease in the
flow capacity of the Drainage Canal above Sag; and in the Chicago River.

Further improvement in the Chicago River and in the Drainage Canal be-
tween Damen Avenue and Summit could be easily accomplished by dredging
the central 100 feet of the cross-section to a greater depth.

The improvement of these channels, as indicated, would involve the exca-
vation of approximately 7,200,000 cubic yards of rock and about 4,300,000
cubic yards of glacial drift, or earth, at a cost of about $30,000,000. A more
detailed study would result in a better hydraulic balance between the various
channels and sections of channels.

Efforts 1955 to 1957

Since 1955, when the storm water drainage program within The Sanitary
District of Chicago, outside the City of Chicago, then estimated to cost
$72,000,000, was submitted to the Illinois General Assembly, little has
happened to change the picture.

The Metropolitan Sanitary District of Greater Chicago has caused the
dredging of 117,325 cubic yards of material from the North Branch of the
Chicago River and the North Shore Channel, at a cost of $378,960. This in no

way enlarged this channel, but such maintenance dredging did restore the original cross-section and improved conditions for flood flow.

Currently, maintenance dredging of the North Shore Channel, from Foster Avenue to Wilmette, is under way. Some 250,000 cubic yards of silt and fill are to be removed, during the year 1958, at a cost of about $1,110,000, to restore the balance of the North Shore Channel to its original cross-section.

Debris and silt and some shrubbery were removed from the channel of the Des Plaines River between the old Lockport Controlling Works and Ninth Street bridge, in Lockport, and the original channel restored in 1956. Similar work is now under way between the Ninth Street bridge and the Elgin, Joliet, and Eastern Railway bridge. This will be finished in 1958.

Three additional sluice gates at the old Lockport Controlling Works were equipped with operating machinery, 1956-1957, thus adding about 7,500 cfs to the discharge capacity from the Main Drainage Canal, at Lockport. Currently, studies are being made of methods to obtain 10,000 to 12,000 cfs more outflow capacity, probably at the Lockport Power House. Outflow capacity was somewhat inadequate during the October 1954 flood, when two of the sluice gates were blocked by some loose barges.

The channel of Deep Run is being enlarged between Sixteenth Street bridge, in Lockport, and its junction with the Main Drainage Canal below the Lockport Power House. The improved channel will have a storm flow capacity of 6500 cfs. This work will be finished in 1958, at a cost of about $500,000, and should alleviate the worst flood conditions in Lockport, Illinois.

The City of Chicago continued construction on its long range program of Auxiliary Outlet Sewers, presumably at a rate of about $15,000,000 per year. A bond issue of $8,000,000, for this work, was approved by the voters, in 1957, leaving about $70,000,000 of the future financing yet to be approved.

Estimates of Cost of Future Work

So little has been done toward the alleviation of flood conditions in Cook County, outside the City of Chicago, that the estimates of costs, submitted in 1955, may well be repeated, with minor modification. One important and costly item, however, must be added, namely, the enlargement of the Main Outlet Channel, at $30,000,000.

The entire future storm drainage program, not yet financed, is outlined in the tabulation following.

Storm Water Drainage Program - February 1958:

Enlargement of Main Drainage Canal $30,000,000

Improvement of Des Plaines River, from
 Riverside to north line of Cook County 9,000,000

Improvement of North Branch of Chicago River,
 north of Lawrence Avenue 4,000,000

Enlargement of North Shore Channel 2,000,000

Enlargement of Stony Creek, and construction
 of tributary ditches and sewers 8,000,000

Improvement Calumet Union Drainage Ditch 2,000,000

Improvement Feehanville and Weller Ditches 1,000,000

Improvement Thorn Creek 1,000,000

Improvements along Addison and Silver Creeks 4,000,000

Storm water outlets for Posen, Midlothian,
  Markham, South Blue Island 4,000,000

Storm water outlets, etc. for villages in
  Grand Calumet and Little Calumet areas 6,000,000

        Total Cook County, outside Chicago $72,000,000
        Total City of Chicago, Sewers 70,000,000

              Total storm water outlets $142,000,000

Storm water sewer systems for the various villages in Cook County, esti-
mated 1955 to cost $31,000,000, are not included in the above estimate. The
improvements of Salt Creek, in DuPage County, estimated 1955 at $5,000,000,
is of some importance to the Chicago area, but is omitted.

CONCLUSIONS

From a study of floods in the Chicago area, it is obvious that the present
outlet channels are not adequate to handle the storm runoff during a great
flood. No change has been made in the Main Outlet, that is, in the Drainage
Canal between Sag and Lockport, since 1900. In that time, the storm runoff
from the Chicago and Calumet areas has probably more than tripled. Local
drainage has been and is being improved in many parts of the area, but these
improvements can not be fully effective until the Main Outlet is made adequate.
Therefore, the enlargement of the Drainage Canal might be considered as
of first importance. This will be particularly true of the stretch of this canal,
downstream from Sag, since the widening of the Calumet-Sag Channel will, in
the next four years, cause a tremendous increase in storm flow from the
Calumet region. Enlargement of the Drainage Canal is estimated to cost
$30,000,000, or about the amount of damage estimated from one great flood,
in October 1954. Damage from the flood of July 1957 was less, but a figure of
$10,000,000 has been mentioned. It is probable that the cost of flood pro-
tection would be less than the cost of flood damage.
All the projects listed in the program above, except the enlargement of the
Drainage Canal, are urgently needed for protection against the ordinary flood,
or that due to a ten-year storm. Residents of some of the communities in-
volved were in pitiful distress during the great floods of 1954 and 1957.
It is not the purpose of this paper to suggest specific improvements for
handling storm water flow in any part of the Chicago area; but rather to out-
line the problem and furnish background material and factual data which will
assist future students of the subject in arriving at definite recommendations.
Proper solution of the problem will require a construction program far beyond
the imagination of everybody except a few who have been close enough to it to

realize its magnitude. Considerable time, at least ten years, will be required for the physical construction, in an orderly fashion, of the needed works, under any method of financing which will not be too burdensome. It is a program which must necessarily be carried out on an area-wide basis.

Chicago is a great city, although it is a young city, only 120 years old. Perhaps it has grown too fast and some of its problems have been neglected. It can become the greatest city in the world; and the solution of the storm drainage problem, now, would enhance that prospect. If the powers that be will face that essential issue, and act, future generations will bless them. If they procrastinate and postpone such really necessary improvements, future generations will condemn them for lack of courage. It is well known that indecision is the greatest failing of the average office holder; but there is no time for indecision in a matter so important to a large and populous community as the prevention of flooding. The longer such matters are deferred, in a growing community, the greater the cost will be.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

HYDRAULIC ANALYSIS OF SURGE TANKS BY DIGITAL COMPUTER

Nicholas L. Barbarossa,[1] M. ASCE

SYNOPSIS

It is shown how certain surge problems have been successfully adapted for solution by an IBM 650 computer. Machine results for the Oahe surge system are presented and compared with those obtained by independent manual methods of arithmetic integration. Machine-computation time for each of the cases discussed averaged between one to one and one-half hours. Sufficient data are given to permit individual checks on machine analysis and computations in a relatively short time.

Computer results are discussed to indicate that treatment of the total problem of transients as one of separable parts in the analysis of surge-tank stability by conventional methods leads to inconclusive results. It appears that a comprehensive study by digital computer of the total problem of transients is feasible, but a machine programming job of great scope will be required before fruitful results are available.

INTRODUCTION

High-speed, automatic digital computers have the potentiality of opening up new horizons of understanding of problem phenomena. Machine solutions will often provide a broader perspective of problem phenomena than can be feasibly obtained by manual computations because of the relative rapidity with which the effects of various factors and assumptions involved in a problem can be tested and explored. By demonstrating how this advantage of a digital computer has been actually utilized for solving surge problems in a particular case, additional evidence is provided of the growing usefulness of computers in engineering work.

     The study discussed herein is concerned with mass-oscillation phenomena in a surge system, which includes a pressure conduit and two restricted-orifice type surge tanks in tandem independently connected to the main conduit (Fig. 1). The system supplies water to a reaction-type turbine. Waterhammer effects are not included in the study. Thus, the study is primarily concerned with the solution of surge-tank problems, involving load rejection, load demand, and hydraulic stability of the tanks under small load changes at minimum head.

     Given the initial-flow conditions and all other pertinent data, the values of various discharges and heads as a function of time for the unsteady flow conditions within the surge system, resulting from power-load changes, may be determined by conventional methods of arithmetic integration.[1,2]* Application of these methods to surge problems is based upon the fundamental assumption that the compressibility of water and the elasticity of flow boundaries may be ignored. This means that pressure changes in the surge system are transmitted with infinitely high velocities and that water columns in different parts of the system may be regarded as solids in evaluating positive or negative accelerative effects. The same fundamental assumption is used as a basis for development of machine methods. However, since a digital computer is to be employed to obtain solutions, there is no need to make as many other simplifying assumptions as are ordinarily dictated by the time limitations of manual solutions. For example, acceleration heads within the risers, variation of friction factors with changing Reynolds Number, tee losses, and finer time increments in the step-by-step process may all be introduced with only comparatively small increase in the machine time required for problem solutions.

     · A list of the physical relationships required for machine solution of surge problems associated with the system depicted on Fig. 1 and the sign convention that has been used are given in Appendix III.

Basis of Analysis and Computation

Head and Discharge Relationships

     Based upon the fundamental assumption of instantaneous transmission of pressure changes, application of the energy principle to the upstream conduit (Fig. 1) for any instant of time yields**

$$h_{a_1} = E_R - P_1 - h_{L_1} \tag{1}$$

The velocity head for the upstream conduit is included in h_{L_1} for convenience; also included are all form and surface-resistance losses between the reservoir and point (1). Relationships similar to Eq. (1) for other parts of the surge system are given in Appendix III.

     All acceleration heads are defined by the conventional, one-dimensional formula

$$h_a = \frac{L}{gA} \frac{dQ}{dt} \tag{2}$$

---

*The numbers in parentheses indicate references listed in Appendix II.
**The symbols used are defined in Appendix I.

DIVERGENT FLOW

CONVERGENT FLOW

OTHER CASES
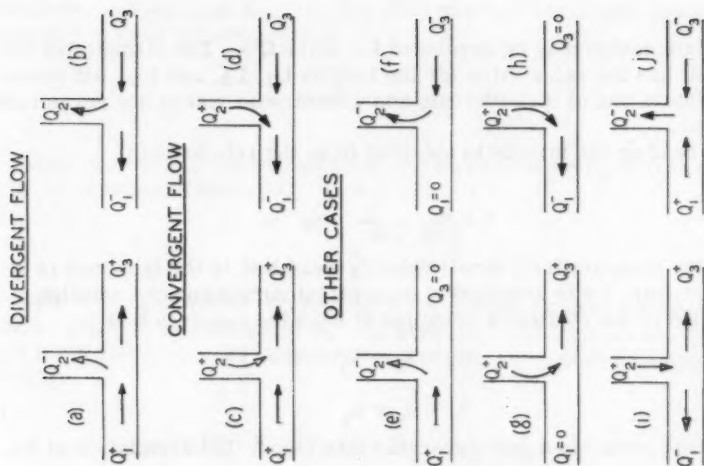
Note: Although indicated flow patterns apply to upstream tee, similar patterns apply to downstream tee.

FIGURE 2 - FLOW AT THE TEES

FIGURE I - DEFINITION SKETCH
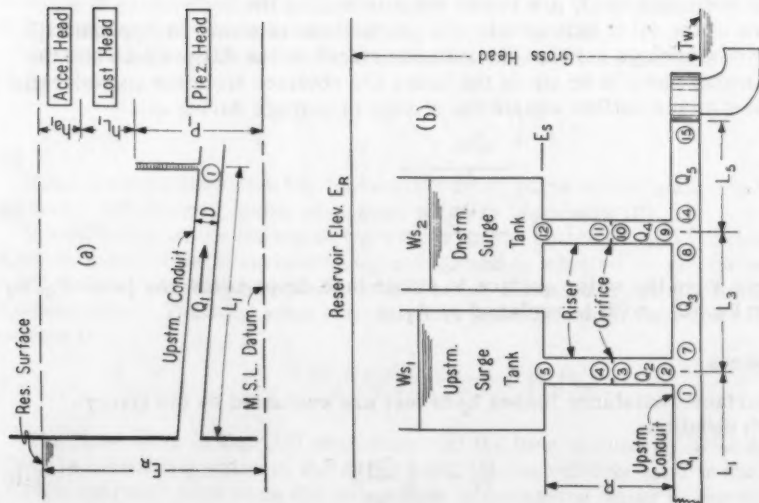
The sign of h_a will depend upon the sign of the derivative $\frac{dQ}{dt}$.

By appropriate use of Eq. (2), Eq. (1) may be written in the form

$$\frac{dQ_1}{dt} = \frac{gA}{L_1}(E_R - P_1 - h_{L_1})$$ (3)

Similar relationships may be developed for other Q's. The diameter of the main conduit has the same value for the lengths L_1, L_3, and L_5. All corresponding dimensions of the upstream and downstream risers and surge tanks are the same.

The net head on the turbine is obtained from the relationship,

$$H = P_{15} + \frac{V_5^2}{2g} - T_w$$ (4)

In Eq. (4), the comparatively small velocity-head lost in the tail-race is ignored. Moreover, Tw is assumed to be constant during any one solution.

Application of the continuity principle at each tee requires that

$$Q_1 + Q_2 = Q_3$$ (5)

$$Q_3 + Q_4 = Q_5$$ (6)

at each instant, even when flow reversals take place. Differentiation of Eq. (5) gives

$$\frac{dQ_1}{dt} + \frac{dQ_2}{dt} = \frac{dQ_3}{dt}$$ (7)

A similar expression may be obtained by differentiating Eq. (6). Equations of the form indicated by (7) are useful for eliminating the derivatives from equations of the form indicated by (3). Actual use is shown in Appendix III.

The relationships between the water surfaces in the surge tanks and the instantaneous flows in or out of the tanks are obtained from the consideration that inflow minus outflow equals the change in storage during dt:

$$\frac{d(Ws_1)}{dt} = -\frac{Q_2}{A_f}$$ (8)

$$\frac{d(Ws_2)}{dt} = -\frac{Q_4}{A_f}$$ (9)

For cases when the water surface in either tank drops below the level E_s, A_f in either Eq. (8) or (9) is replaced by A_r.

Head Losses

All surface-resistance losses h_f in feet are evaluated by the Darcy-Weisbach equation,

$$h_f = f\frac{L}{D}\frac{|Q|Q}{2gA^2}$$ (10)

This equation is applied not only to the main conduit but also to the risers. Surface-friction losses within the surge tanks are ignored because, ordinarily, they are negligibly small.

In some installations a part of the total length L_1 of the upstream conduit will be lined with concrete and the remainder with steel. Provision for this has been made in the machine program. Let L_c = length that is lined with concrete and L_s = length that is lined with steel, so that $L_1 = L_c + L_s$. Designating f_c and f_s as the friction factors for the concrete and steel lengths, respectively, and utilizing Eq. (10), the total surface-resistance losses from the reservoir to point (1) are given by

$$h_{f_1} = (f_c L_c + f_s L_s) \frac{|Q_1| Q_1}{2gDA^2} \tag{11}$$

Except for the tee losses, all form (or shape) losses h_s in feet are evaluated by an equation of the type,

$$h_s = K \frac{|Q| Q}{2gA^2} \tag{12}$$

in which K is assumed to be constant for a particular flow-boundary configuration. Eq. (12) indicates that h_s will have the same sign as Q. The total head loss* due to various boundary forms from the reservoir to point (1) is given by

$$h_{s_1} = (K_v + K_{tr} + K_e + K_s + K_{ts} + K_{hb} + K_{vb} + K_{ot}) \frac{|Q_1| Q_1}{2gA^2} \tag{13}$$

Strictly, all K's in Eq. (13) will have one set of values when Q_1 is positive and will have a different set of values when Q_1 is negative (flow toward the reservoir). In this study the K's are assigned one set of values and are assumed to remain constant during any one machine run. (If deemed necessary, the machine program can be modified to allow the K's to vary with the direction of Q_1 and also with the magnitude of the velocity head.)

Combining Eqs. (11) and (13), the total head loss from reservoir to point (1) is

$$h_{L_1} = \left[K_1 + K_1' (f_c L_c + f_s L_s)\right] |Q_1| Q_1 \tag{14}$$

Head losses (other than tee losses) for other parts of the main conduit are similarly determined, using analogous notation (Appendix III).

In evaluation of the losses in the risers and at points (5) and (12), attention must be given to the direction of Q_2 and Q_4 and to whether or not the water surfaces Ws_1 and Ws_2 are above or below the elevation E_s at any instant under consideration. The total head loss for the upstream surge tank and riser system is

$$h_{L_2} = (C_r + K_2 f_2 + \frac{1}{2gC_o^2 A_o^2}) |Q_2| Q_2 \tag{15}$$

The first term in Eq. (15) evaluates: (a) the loss of velocity head when the riser is delivering water to the surge tank; (b) the velocity head in the riser and the entrance loss when the surge tank is delivering water to the riser. The second term in Eq. (15) evaluates the surface-friction loss in the riser.

---

*Strictly, h_{s_1} is more than a head loss, since it includes the velocity head in the upstream conduit. K_v is equal to one in all cases.

Eqs. (III-27) and (III-28) in Appendix III indicate how C_r and K_2 are evaluated. In the two expressions for K_2 (Eq. (III-28)) allowance is made as to whether or not the total friction boundary has a length R or something less than R. The third term in Eq. (15) determines the head drop across the orifice and is based upon the standard-orifice formula. In this study C_O is taken as being constant during any one machine run. Actually, even if the orifice may be considered a thin-plate orifice as such, it is likely that C_O will vary as Q_2 changes owing to the condition that the orifice plate is probably not sufficiently distant from major flow disturbances in most installations. However, C_O may be easily introduced into the machine program as a function of appropriate variables, if desired and if data are available for realistic evaluation. Otherwise, C_O may be varied from one run to another to see what effect different values have on the hydraulic transients, as has been done in the Oahe study. Head losses h_{L_4} in the downstream riser and surge tank are obtained in the same manner as used to obtain h_{L_2}.

Tee Losses

In determining the tee losses at any instant of time, the three Q's at each tee must be known in both magnitude and direction at that instant. Evaluation of the tee loss will then depend upon which of the possible flow patterns indicated on Fig. 2 applies at that instant. Provisions have been made in the machine program for evaluating the tee losses at each instant of time for all flow patterns shown on Fig. 2 except the last two (i and j). (When flow patterns 2(i) and 2(j) occur, the tee loss is assumed to be zero.) For these two cases, it is uncertain that appropriate data for evaluation exist. Should data be available, the program could be readily modified to accommodate these two cases. Data are available[4] to evaluate cases (a) through (h) on Fig. 2. Obviously, cases (e) through (h) are special cases of (a) through (d).

The tee losses T in feet are determined by equations of the general form,

$$T = C_t \frac{v^2}{2g} \tag{16}$$

wherein, C_t is a variable coefficient, which depends upon the discharge ratios $\frac{Q_2}{Q_1}$ or $\frac{Q_2}{Q_3}$ at the upstream tee ($\frac{Q_4}{Q_3}$ or $\frac{Q_4}{Q_5}$ at the downstream tee) and upon whether the flow is divergent or convergent. The velocity head to be inserted in Eq. (24) is either $\frac{V_1^2}{2g}$ or $\frac{V_3^2}{2g}$ at the upstream tee ($\frac{V_3^2}{2g}$ or $\frac{V_5^2}{2g}$ at the downstream tee) depending upon whether the main conduit flow is directed toward (+) or away (-) from the turbine. In Eq. (16) C_t is replaced by C_D and C_D' for divergent flow and by C_c and C_c' for convergent flow. Four sets of coefficients are required in order to obtain the piezometric-head changes at each tee for all cases considered. (Table 5 has been utilized for numerical evaluation when $D_r/D = 2/3$.) The coefficients include allowances for changes in velocity head from conduit to riser and vice versa. The details of tee-loss determination are given in a summarization in Appendix III.

With reference to Fig. 1, if the tee losses are ignored, the piezometric heads at points (1), (2), and (7) are assumed to be equal; that is, $P_1 = P_2 = P_7$. These three heads differ only when the tee losses are introduced. Thus, for example,

$$P_1 - P_2 = T_{1,2} = C_D \frac{V_1^2}{2g} \qquad (17)$$

$$P_1 - P_7 = T_{1,7} = C_D' \frac{V_1^2}{2g} \qquad (18)$$

In both of these equations (applicable to case (a) on Fig. 2) the sequence of writing the subscripts for T is important from the standpoint of avoiding confusion in signs. Thus, $T_{1,2}$ indicates $P_1 - P_2$ and not $P_2 - P_1$. The same reasoning applies to the downstream tee.

In all instances of unsteady flow when the tee losses are taken equal to or are zero, the total energy heads at the tees are not balanced because of the instantaneous difference in velocity heads between main conduit and riser (between points (1) and (2) on Fig. 2, for example). (The number of times that this would occur during one total solution when the tee losses are evaluated at all other instances during that same solution is relatively small.) This unbalanced condition may be assumed to be taken care of by orifice effects in the riser, that is, by the orifice coefficient. If deemed necessary, special provisions can be incorporated in the machine program to take care of the velocity-head differences when $T = 0$. In this study, such provisions have not been made, because over-all results for the Oahe project, at least, would not be significantly affected.

Gate-Time and Turbine-Performance Curves

Transient effects within the hydraulic system are established by turbine wicket-gate movements. In order for the program to work successfully, the gate opening G in per cent must be known at each instant of time during the initial interval of governor action, after which the gate opening is assumed to remain constant. This statement is only partially true for surge-tank stability studies. (Discussion of what is involved in stability studies is presented in another section of the paper.) The gate-time curve is given by

$$G = F_g(t), \qquad (19)$$

in which, F_g = a function. It is not necessary that this be a linear function of time. Any single-valued function of time may be used, and it may include dead and cushion time (for opening and closing of gates, respectively.) Such curves are usually furnished by the manufacturer of the turbine.

In addition, turbine-performance curves are required. They enter the program as a table involving three variables, whose inter-relationship in general mathematical form is expressed as

$$H = F(G, Q_5), \qquad (20)$$

in which F is a function. For carrying out solutions for full-load rejection at maximum turbine design head H_{max} and for full-load demand at minimum turbine design head H_{min}, it will usually be necessary to have tabular values of G and Q_5 for values of H well above H_{max} and of H well below H_{min}. Otherwise, the computer will stop because of artificial bounds imposed by lack of tabular values in regions of computation which the analyst did not foresee or overlooked. (What this means is that when the computer reaches the end of a table and it has no instructions on what to do next, it stops.)

To perform surge-tank stability studies along conventional lines, the following set of turbine curves is utilized instead of the family of curves indicated by Eq. (20);

$$H = F_q(Q_5)_{Bhp} = \text{Constant} \tag{21}$$

in which F_q is a function and Bhp = brake horsepower. This equation simply gives the relationship between H and turbine discharge Q_5 when the turbine power output is constant and can include the effects of varying efficiency. The use that has been made of such curves and the difficulties experienced in the computer study are discussed below.

Machine Computation

With the aim of keeping machine-programming details to a practical minimum herein, Fig. 3 has been especially prepared for the purposes of this paper. The chart simply depicts the main elements of problem solution but not the actual sequence of machine operations.* To show the actual sequence of machine operations would require considerable detail, the inclusion of which would tend to obscure the main elements of the problem. Fig. 3 applies to problem solutions that start from steady-state conditions, but the detailed machine program is not so limited. The equation numbers shown in some of the boxes refer to the relationships given in Appendix III. It is suggested that the reader first examine that appendix before proceeding with the remainder of the paper.

In box 6 (Fig. 3) all lost heads are computed from the steady-state $Q = Q_1 = Q_3 = Q_5$ and data given in boxes 2, 3, and 4, and tailwater level is adjusted to the value it must have for all heads to balance exactly within the machine for the given reservoir elevation E_R (box 2). Alternatively, the tailwater level may be kept constant, and E_R may then be adjusted for balancing of steady-state heads. This balancing process insures that the computer starts off with a totally consistent set of values for the variables involved.

Once steady-state conditions are balanced within the machine, a transient is set up by applying the initial time interval Δt (box 5), which fixes the new gate setting from $G = F_g(t)$ in box 4. The new G in turn fixes the new H from the turbine-performance curves (box 10), using the value of Q_5 at the beginning of the time interval. As a result, all other variables are changed, and the machine will compute their values at each step as the transient develops. The initial Δt is arbitrarily selected by the analyst and can be as small as seems warranted. Values as low as 0.01 second have been used to start the transient; in most cases, initial $\Delta t = 0.10$ has been used. The computer can be instructed through control information (box 5) to use Δt values of 0.1 up to a total time of $t = 1.00$ and to use time intervals of one second thereafter up to, say, 80 seconds, after which it would use 5-second intervals until the total time for the machine run (given in box 5) is reached. These figures are merely illustrative. Any reasonable set of time intervals may be used, including possible use of a constant Δt to the end of the run. However, the method of solving differential equations (discussed below) will not work expeditiously if the assigned time intervals for step solutions are too large.

---

*Complete details, including all flow charts and the machine codes, are given in the machine program itself, which is being prepared as a separate document for distribution on a limited basis.

```
                          ┌─────────┐
                          │  Start  │
                          └────┬────┘
                               │
                          ┌────┴──────────┐  ①
                          │ Read in initial│
                          │ program instructions│
                          └────┬──────────┘
```

① Read in initial program instructions

② Read in all physical data: L_c, L_u, L_g, L_d, R C_g, D, D_r, D_f, D_g, r, K_{rf}, K_e, K_o, K_{te}, K_{bb}, K_{eb}, K_{ot} E_o, E_g, and K_e' for Surge Tanks.

③ Compute K_1, K_1', A, A_r, A_c, A_g, $L_1 = L_c + L_u$, r_1, r_3, r_2, $\dfrac{1}{2gC_o{}^2 A_o{}^2}$

④ Read in main program instructions and all tables: $f_c = F_c (N_R)$, $f_r = F_r (N_R)$; Tables for f_2, f_3, f_4, f_5, if different; Table for Tee Losses; $H = F(G, Q_B)$; $G = F_2 (t)$

⑤ Read in control information: Tee losses computed or ignored? Riser acceleration computed or ignored? initial time interval Δt; Total time for run; initial values of Q_1, Q_2, Q_3; Values of G at beginning and end of governor action.

⑥ Compute h_{L_1}, h_{L_2}, h_{L_3}, W_{o_1}, W_{o_2}, and adjust Tailwater to conform for steady state, t=Q

⑦ Derivative Computation

⑧ Is G=0? — Yes → ⑨ $r_3 = 0$, $Q_3 = 0$
                — No → ⑩ Compute H from H=F(G, Q_B)

⑪ Compute Q_2 & Q_3

⑫ Compute V_1, V_2, V_3, r_3, r_2, friction f_c, f_r, f_3, f_4, f_5

⑬ Compute: h_{L_1} from eq. III-21,22 h_{L_2} from eq. III-26,27,28 h_{L_3} from eq. III-24 h_{L_4} from eq. III-29,30,31 h_{L_5} from eq. III-25

⑭ $\dfrac{d(W_{o_1})}{dt}$ from III-43, $\dfrac{d(W_{o_2})}{dt}$ from III-44

⑮ Tee Losses $T_{1,2}$, $T_{1,7}$, $T_{8,9}$, $T_{8,16}$ from eq. III-32 through III-39

⑯ Riser Acceleration — No → ⑲ $P_1 = W_{o_1} - h_{L_1} - T_{1,2}$, $P_3 = W_{o_2} - h_{L_4} - T_{8,9}$
                        — Yes → ⑰ $P_1 = \dfrac{v - \delta}{\delta - \phi}$ Eq. III-20

⑱ $P_6 = \phi P_1 + v$ Eq. III-17

⑳ $P_7 = P_1 - T_{1,7}$, $P_{10} = P_6 - T_{8,16}$ Eq. III-12, 13

㉑ $P_{15} = Tw + H - h_{L_5}$ Eq. III-47

㉒ $h_{a_3} = P_{10} - P_{15} - h_{L_4}$ $\dfrac{dQ_3}{dt} = gr_3 h_{a_3}$; eq. III-3

㉓ $h_{a_2} = P_9 - P_7 - h_{L_3}$ $\dfrac{dQ_2}{dt} = gr_2 h_{a_2}$; eq. III-2

㉔ $h_{a_1} = E_a - P_1 - h_{L_2}$ $\dfrac{dQ_1}{dt} = gr_1 h_{a_1}$; eq. III-1

㉕ Solution of Differential Equations in boxes ⑭, ㉒, ㉓, ㉔, by small time increments Δt and automatic error control.

㉖ Print Out t, Q_1, h_{L_1}, h_{a_1}, P_1, $T_{1,7}$, $T_{1,2}$, W_{o_1}, h_{L_2}, P_9, Q_2, h_{L_3}, h_{a_2}, Q_6, W_{o_2}, h_{L_4}, $T_{8,9}$, $T_{8,16}$, $\dfrac{V_2}{2g}$, h_{L_5}, h_{a_3}, H, G, Q_3
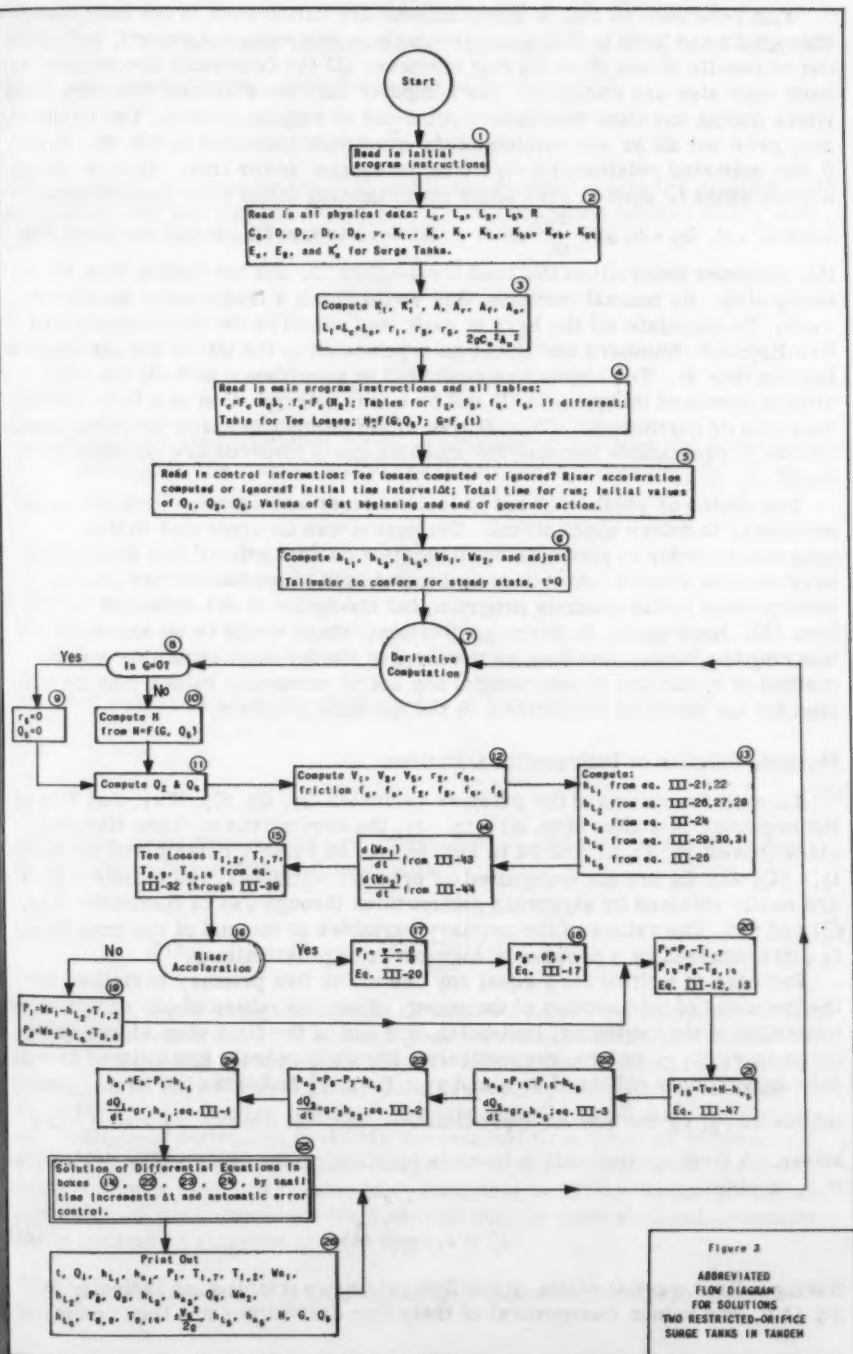
Figure 3

ABBREVIATED FLOW DIAGRAM FOR SOLUTIONS TWO RESTRICTED-ORIFICE SURGE TANKS IN TANDEM

With reference to Fig. 3, computations are carried out in the loop, boxes 7 through 25 and back to 7 (but not precisely in the sequence shown), with print-out of results in box 26 occurring whenever all the conditions for solution for each time step are satisfied. The computer may work around this loop many times during any time step before print-out of results occurs. The computer may print out all or any number of the quantities indicated in box 26. In box 9, the indicated relationship $r_5 = 0$ is, of course, never true. This is simply a trick which is used to save some programming steps for rejection cases when $G = 0$, $Q_5 = 0$, and $\dfrac{dQ_5}{dt}$ must equal zero in box 22. It will be noted that the computer determines the head losses (box 13) and tee losses (box 15) at every step. By manual methods, this would entail a tremendous amount of work. To calculate all the h_L's at each step requires the determination of five Reynolds Numbers and linear interpolations in the tables for six friction factors (box 4). Tee losses are evaluated in accordance with all the conditions specified in Appendix III and by linear interpolation in a table having four sets of coefficients. This degree of refinement in surge-problem computations is practicable because the machine costs involved are comparatively small.

The choice of whether or not to include riser acceleration (box 16) is not necessary in future applications. The option was incorporated in this program in order to give a direct indication of what effects this item would have on final results. An option on computation of tee losses has also been incorporated in the machine program, but the option is not indicated on Fig. 3 (box 15). Here again, in future applications, there would be no necessity for ignoring tee losses. As long as the form of the tee-loss equations and the method of evaluation do not change, any set of numerical values may be utilized for the tee-loss coefficients in the machine program.

Machine Solution of Differential Equations

Knowing the values of the primary variables Q_1, Q_3, Q_5, Ws_1, and Ws_2 at the beginning of a time step, $\Delta t = t_2 - t_1$, the derivatives of these five variables (boxes 14, 22, 23 and 24 in Fig. 3) may be readily determined for time t_1. (Q_2 and Q_4 are not designated as primary variables, since their values are easily obtained by algebraic subtractions through use of continuity Eqs. (5) and (6)). The values of the primary variables at the end of the time step, t_2, are obtained by a process of high-order approximations.[10]

For ease in writing let y equal any one of the five primary variables for the purposes of this section of the paper. Then the values of any one of these quantities at the beginning, mid-point, and end of the time step Δt are designated as y_1, $y_{1.5}$, and y_2, respectively. Roman numerals are utilized to designate approximate values of $y_{1.5}$ and y_2. Thus, y_2^{I} indicates the first approximation of y_2, y_2^{II} the second approximation, etc. At time t_1, y_1 and $\dfrac{dy_1}{dt}$ are known. A first approximation is made by utilizing the value of the derivative at t_1 to yield

$$y_2^{I} = y_1 + \frac{dy_1}{dt}\,\Delta t.$$

(22)

Having a first approximation of the five primary variables, as indicated by Eq. (22), new values (temporary) of their five derivatives are then computed

exactly the same way as was done at time t_1; these new derivatives are designated $\dfrac{dy_2^{I}}{dt}$. The next step yields a better approximation of y_2 by making use of the average of the two derivatives now at hand, so that

$$y_2^{II} = y_1 + \frac{\Delta t}{2} \left(\frac{dy_1}{dt} + \frac{dy_2^{I}}{dt} \right). \tag{23}$$

Having set y_2^{II} as given by Eq. (23) aside, the computer cuts the original time interval in half and determines $y_{1.5}^{II}$ in exactly the same manner as y_2^{II} was calculated, with the result that

$$y_{1.5}^{I} = y_1 + \frac{dy_1}{dt} \frac{\Delta t}{2}, \text{ and} \tag{24}$$

$$y_{1.5}^{II} = y_1 + \frac{\Delta t}{4} \left(\frac{dy_1}{dt} + \frac{dy_{1.5}^{I}}{dt} \right) \tag{25}$$

Still another approximation for y_2 is obtained by applying the same method of computation to the second half of the time interval (from $t_{1.5}$ to t_2), so that

$$y_2^{III} = y_{1.5}^{II} + \frac{dy_{1.5}^{II}}{dt} \frac{\Delta t}{2}, \text{ and} \tag{26}$$

$$y_2^{IV} = y_{1.5}^{II} + \frac{\Delta t}{4} \left(\frac{dy_{1.5}^{II}}{dt} + \frac{dy_2^{III}}{dt} \right). \tag{27}$$

The final values of all primary variables at time t_2 are taken as*

$$y_2 = y_2^{IV} + \frac{y_2^{IV} - y_2^{II}}{3} \tag{28}$$

But before these final values are assigned to y_2, relative error tests are made. The relative error is defined as

$$R_y = \frac{y_2^{IV} - y_2^{II}}{3y_2^{IV}}. \tag{29}$$

If $\left| R_y \right| > \epsilon_1$, a pre-assigned margin of relative error, the computer cuts the original time interval Δt in half and goes through the whole procedure (embodied in Eqs. (22) through (29)) all over again. The machine will repeat this process until $\left| R_y \right| < \epsilon_1$. (For the Oahe project, ϵ_1 was given a value of 10^{-3}.) To prevent the computer from taking unnecessarily small time increments, the machine is instructed to double the original time interval whenever $\left| R_y \right| < \epsilon_2$, another pre-assigned margin of relative error. (For the Oahe project ϵ_2 was assigned a value of $\epsilon_1/100$, that is 10^{-5}.) By the procedure just outlined, solutions are advanced time step by time step and automatic error control is achieved at each step.

---

*The factor three in Eq. (28) is based upon theoretical considerations.[10]

The necessity for such high-order approximations for solutions of differential equations has not been investigated, because the machine time required per total solution is reasonable and because office time could not be spared for the mathematical explorations that would be required. It is believed that the machine method of solving differential equations for surge problems, as discussed herein, can also be applied profitably to machine solutions of a variety of other engineering problems.

Application of Machine Methods to Oahe Surge System

Conditions and Data for Numerical Solutions

Oahe Dam, located about six miles upstream of Pierre, South Dakota, is a key project of the system of multiple-purpose reservoirs on the Missouri River.[5,6] Each of the scheduled seven generator units at Oahe will have a nameplate capacity of 85,000 kilowatts, and hydro-power will be delivered to the generators by Francis-type turbines. The hydraulic surge system pertaining to any one of the seven units is schematically shown on Fig. 1. The length of the upstream conduit L_1 is different for each unit due to physical layout. Machine results have been obtained only for unit number 6, for which $L_1 = 3,820$ feet in all cases, except cases Ie and IIe in Tables 6 and 7. These two tables and Table 8 give the initial conditions (steady state) and other pertinent data upon which manual and all machine computations for full-load rejection, full-load demand, and surge-tank stability analysis are based. References to manual computations in all tables and figures herewith apply to those made independently by engineers of Sverdrup and Parcel, Inc., a firm under contract with the Omaha District, Corps of Engineers, for design of the Oahe power plant and surge tanks.

All computations are based upon: surge tank diameters of 70 feet with no interconnection between tanks; riser diameters of 16 feet; main conduit and penstock diameter of 24 feet; riser height R of 81 feet; orifice diameters of 12.33 feet; a distance between risers L_3 of 40 feet (except for cases Ie and IIe); and other data given in Tables 6, 7, and 8. With $L_3 = 40$ feet, the surge tanks will, of course, not fit concentrically over the risers. Hence, for cases Ie and IIe the distance L_3 was doubled in order to permit a concentric arrangement of surge tanks and risers, if desired. From a hydraulic stability standpoint, the distance between tanks should probably be as small as practicable. The eccentric arrangement with $L_3 = 40$ feet has been adopted for the Oahe project.

All friction factors for both manual and machine computations have been evaluated from data given in Tables 1, 2, and 4. When $N_R < 10^5$, friction losses for the smallest diameter (16 feet) are negligibly small, and hence, no values of f are given in the tables for $N_R < 10^5$. Table 5 has been used to evaluate all tee losses for computer solutions. Manual computations in all cases are based upon ignoring acceleration effects in the risers and in the length L_5, but tee losses have been evaluated using Vogel's data.[4] It will be noted from Tables 6, 7, and 8 that L_1 for manual computations was 3,580 feet instead of 3,820 feet and that a few other items for these computations differ slightly from those used for machine solutions. The reason for this is that computer runs were made at a much later data, after some Oahe design changes had been effected. Other conditions and data as well as results are discussed below.

TABLES 1, 2 & 4

Friction Factors f

TABLE 1*		TABLE 2**		TABLE 4†	
Reynolds Number N_R	f	Reynolds Number N_R	f	Reynolds Number N_R	f
10^5	0.0162	10^5	0.0186	10^5	0.0196
3×10^5	0.0144	2×10^5	0.0167	2×10^5	0.0186
10^6	0.0118	3×10^5	0.0158	3×10^5	0.0176
3×10^6	0.0098	5×10^5	0.0147	5×10^5	0.0165
5×10^6	0.0091	7×10^5	0.0143	7×10^5	0.0161
10^7	0.0082	10^6	0.0138	10^6	0.0158
2×10^7	0.0076	2×10^6	0.0132	2×10^6	0.0155
3×10^7	0.0070	4×10^6	0.0128	4×10^6	0.0152
4×10^7	0.0067	6×10^6	0.0127	6×10^6	0.0152
6×10^7	0.0064	10^7	0.0125	10^7	0.0152
10^8	0.0059	10^8	0.0125	10^8	0.0152

* From Von Karman-Prandtl Curve for hydrodynamically smooth boundaries.

** Based upon Relative Roughness $e/D = 1.25 \times 10^{-4}$

† Based upon Relative Roughness $e/D = 3.3 \times 10^{-4}$

Values in Tables 2 and 4 taken from U. S. Bureau of Reclamation Monograph #7, Figure 4.

ABBREVIATED TABLE 3

OAHE POWER PLANT TURBINE PERFORMANCE

Values of Discharge in C.F.S. Versus Gate Opening and Net Head

G-Gate Opening in %	Net Head on Turbine H in Feet							
	70	110	120	130	160	185	205	240
0	0	0	0	0	0	0	0	0
5	276	370	383	412	470	516	549	588
10	551	760	786	824	940	1031	1097	1176
15	827	1110	1178	1237	1410	1547	1646	1764
20	1102	1480	1571	1649	1880	2062	2194	2351
25	1378	1850	1964	2061	2350	2578	2743	2939
30	1653	2220	2357	2473	2820	3093	3291	3527
35	1928	2580	2749	2886	3290	3609	3840	4115
40	2204	2960	3142	3298	3760	4124	4396	4725
45	2480	3330	3535	3710	4230	4640	4936	5275
50	2800	3705	3910	4110	4665	5082	5398	5800
55	3155	4065	4280	4490	5090	5532	5840	6260
60	3460	4430	4655	4870	5485	5948	6278	6740
65	3765	4775	5010	5230	5895	6322	6682	7235
70	4090	5135	5375	5600	6215	6692	7050	7585
75	4355	5430	5670	5900	6545	7048	7428	8015
80	4600	5750	5985	6215	6875	7390	7778	8330
85	4900	6035	6285	6525	7190	7705	8095	8715
90	5195	6280	6530	6770	7475	8025	8425	9015
95	5450	6490	6735	6975	7685	8250	8677	9355
100	5705	6670	6910	7160	7875	8450	8905	9665

Adapted from Omaha District Curves, Model 734-H, dtd 20 November 1957.

TABLE 5

Tee Loss Coefficients*

$D_r/D = 2/3$

Discharge Ratio $\frac{Q_{riser}}{Q_{conduit}}$**	Diverging Flow			Converging Flow		
	C_b	C_b'	$C_b - C_b'$	C_c	C_c'	$C_c - C_c'$
0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.1	0.15	-0.24	0.39	0.43	0.50	0.07
0.2	0.45	-0.47	0.92	0.68	0.83	0.15
0.3	0.90	-0.60	1.50	0.86	1.12	0.26
0.4	1.47	-0.69	2.16	0.97	1.38	0.41
0.5	2.19	-0.75	2.94	1.07	1.62	0.55
0.6	3.00	-0.75	3.75	1.10	1.82	0.72
0.7	4.00	-0.77	4.77	1.10	2.00	0.90
0.8	5.20	-0.75	5.95	1.05	2.17	1.02
1.0	7.85	-0.65	8.50	0.70	2.45	1.75
Use For Tee Losses	$T_{1,2} T_{8,9}$	$T_{1,7} T_{8,14}$	$T_{1,2} T_{8,9}$	$T_{1,7} T_{8,14}$	$T_{1,2} T_{8,9}$	$T_{1,7} T_{8,14}$	$T_{1,2} T_{8,9}$

* Prepared by Omaha District, CE, from Vogel's data (4).

** Discharge ratios may be either $\frac{Q_2}{Q_1}$, $\frac{Q_2}{Q_3}$ or $\frac{Q_5}{Q_3}$, $\frac{Q_5}{Q_5}$.

OAHE POWER PLANT SURGE TANK ANALYSIS
Two Restricted-Orifice Surge Tanks in Tandem
TURBINE UNIT NO. 6

Table 6 — FULL LOAD REJECTION

Table 7 — FULL LOAD DEMAND

Table 8 — STABILITY STUDY

Legend: C = Computed   NC = Not computed

Results ...

With ... the turbi... of 203 fe... to 66.3 p... tations) ... is achiev... second. ... interpola... performa... G and H ... 70 and 2...

Figs. ... jection c... surge tan... as a func... Computa... manual c... ly higher... computed... 6 is the ...

Case ...
Case ...
Case ...
Case ...
    wh...

Exam... machine ... value of ... surface c... al compu... substanti... Actual m... aged slig... less typi...

Results f...

With r... the reser... opening t... second (g... governor... is the sa... Manual C... except th... is zero (... (instead ... accelera... computer... c.f.s. (co...

Results for Full-Load Rejection

With reference to Table 6, all cases are based upon rejecting full load on the turbine, 142,000 horsepower output, at a maximum net head on the turbine of 203 feet for steady state. For this condition the gate opening is restricted to 66.3 per cent for all computer solutions (70 per cent for manual computations) in order to avoid overload on the generator, and closure of the gates is achieved in 3.31 seconds (computer) at a uniform rate of 20 per cent per second. Turbine-performance values (Eq. (20)) have been obtained by linear interpolation in a table of the form indicated by Table 3. The actual turbine-performance table that was used in machine computations gave Q_5 values for G and H at closer intervals ($\Delta G = 5$ between 0 and 100, $\Delta H = 5$ feet between 70 and 240 feet).

Figs. 5 and 6 indicate some of the results that were obtained for the rejection cases. Fig. 5 depicts the water surface elevation of the downstream surge tank and the difference in water surface elevation between the two tanks as a function of time for all cases indicated in Table 6. Case I and Manual Computations are based upon the same data and conditions, except that for the manual computations L_1 is over 200 feet shorter, initial Q_5 and G are slightly higher, and acceleration heads in the risers and in the length L_5 are not computed. The basis of computation for all other cases shown on Figs. 5 and 6 is the same as for Case I, except that for:

Case Ib, acceleration heads in the risers have not been computed.
Case Ic, the orifice coefficient is 0.8 instead of 0.7.
Case Id, the orifice coefficient is 0.6 instead of 0.7.
Case Ie, the length L_3 is 80 feet instead of 40 feet, L_1 and L_5 being somewhat smaller as a consequence.

Examination of the curves on Fig. 5 indicates successful application of machine methods to rejection problems. It is noted that: (a) the numerical value of the orifice coefficient has the most pronounced effect upon the water surface elevations in the tanks and (b) in comparison with the results of manual computations, the difference in water surface elevations between tanks is substantially greater for all computer cases during the first quarter cycle. Actual machine-computation time for each of the cases shown on Fig. 5 averaged slightly over one hour. The Q values for Case Id on Fig. 6 are more or less typical of those for all other computer cases indicated on Fig. 5.

Results for Full-Load Demand

With reference to Table 7, all cases are based upon picking up load when the reservoir elevation is at minimum design level ($E_R = 1540$). The rate of opening the wicket gates is linear in all cases, the rate being 20 per cent per second (governor time of 5 seconds) for all except IIa. For Case IIa, the governor time is 10 seconds. Applicability of Table 3 to load-demand cases is the same as for the load-rejection cases previously discussed. Case II and Manual Computations (Table 7) are based upon the same data and conditions except that for the latter: L_1 is over 200 feet shorter; initial steady-state G is zero (instead of 5.0); gate opening at end of governor action is 90 per cent (instead of 100 per cent); tailwater elevation is 1420 (instead of 1421.7); and acceleration heads in the risers and in the length L_5 are not computed. All computer cases are based upon an initial steady-state discharge Q_5 of 390 c.f.s. (corresponding to initial G = 5.0) instead of zero. A comparatively small

FIGURE 5

OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD REJECTION -
TURBINE UNIT NO. 6

RES. ELEV. $E_R = 1645$

INITIAL STEADY STATE

TIME IN SECONDS

FIGURE 6
OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD REJECTION
CASE Id
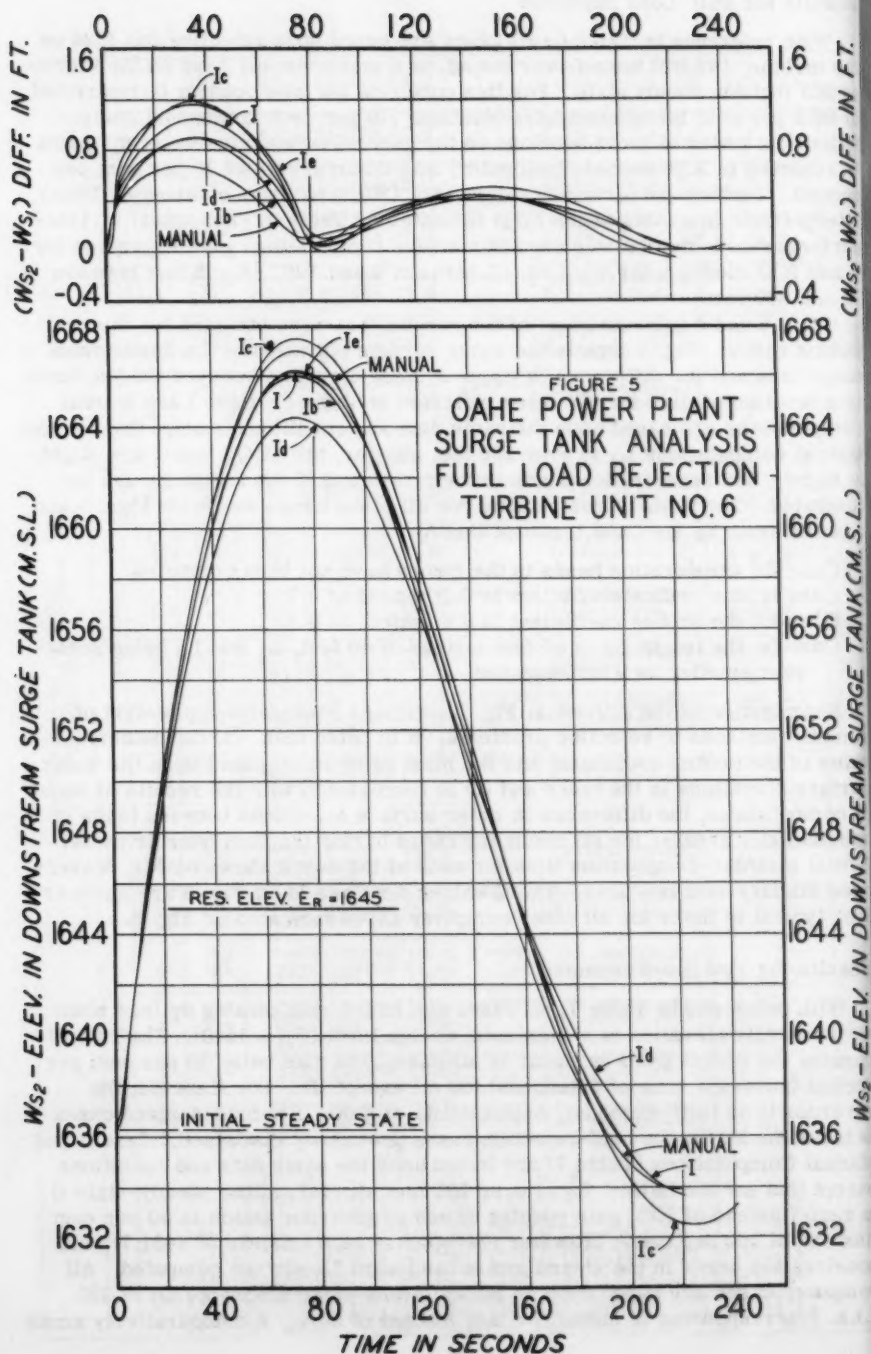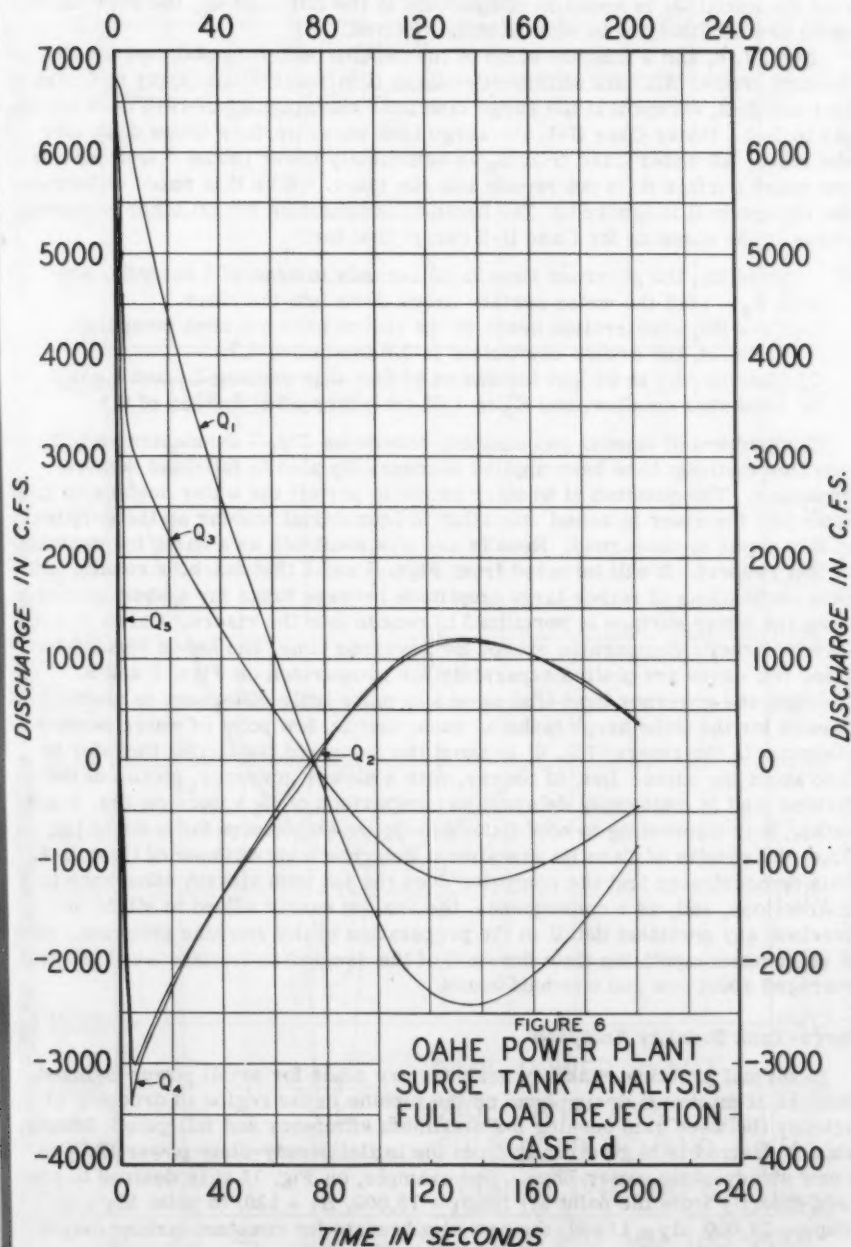
finite value of Q_5 at the start seems to be necessary because of the machine methods of solving differential equations that has been utilized. A smaller value than 390 c.f.s. might have been used successfully. However, provided that the initial Q_5 is small in comparison to the full-load Q_5, the over-all results are not likely to be significantly affected.

Figs. 7, 8, and 9 indicate some of the results that were obtained for the demand cases. All data shown under Case II in Table 7 also apply to Cases II-1 and II-2, except that the surge tank base elevation E_s of 1515 does not apply to II-2. Under Case II-1, the surge tank water surface drops down into the riser, but under Case II-2, E_s is sufficiently lower (about 3 feet) so that the water surface does not recede into the riser. (With this small difference, the change in R is ignored.) The basis of computation for all other computer cases is the same as for Case II-2 except that for:

Case IIa, the governor time is 10 seconds instead of 5 seconds, and with E_s = 1515 the water surface drops down into the riser.
Case IIb, acceleration heads in the risers have not been computed.
Case IId, the orifice coefficient is 0.6 instead of 0.7.
Case IIe, L_3 is 80 feet instead of 40 feet thus causing L_1 and L_5 to be somewhat smaller, and K_e' is 1.00 (re-entry tube) instead of 0.5.

Comparison of manual and machine results on Fig. 7 demonstrates that machine methods have been applied successfully also to full-load demand problems. The question of whether or not to permit the water surface to drop down into the riser in actual operation is immaterial insofar as the purpose of this paper is concerned. Results are now available as a basis for decision in that respect. It will be noted from Figs. 7 and 8 that machine results indicate oscillations of rather large amplitude between tanks for a short period, when the water surface is permitted to recede into the risers. Cases II-1 and IIa are directly comparable except for governor time, and hence results for these two cases are plotted separately for comparison on Figs. 8 and 9. Doubling the governor time (IIa) seems to make little difference in over-all results for the Oahe surge tanks as such; that is, low point of water surface elevation in the risers (Fig. 8) is about the same and the timing therefor is also about the same. But, of course, with a slower governor, pickup of the turbine load is materially delayed, as comparison of Q_5 values on Fig. 9 indicates. It is interesting to note from this figure that except for a slight lag time, the results of Case IIa are almost a carbon copy of those of Case II-1. This demonstrates that the computer does the job with slavish adherence to instructions, and, as a consequence, the analyst cannot afford to slight or overlook any pertinent detail in the preparation of the machine program. Actual machine-computation time for each of the demand cases discussed above averaged about one and one-half hours.

Surge-Tank Stability Analyses

In normal practice, stability analyses are made for small power-demand changes at minimum design head on the turbine in the region of drooping efficiency (between gate opening for maximum efficiency and full gate.) Ideally, what is desired is to go directly from the initial steady-state power Bhp_1 to a new steady-state power Bhp_2. For example, on Fig. 11 it is desired to proceed directly from the point M_1 (Bhp_1 = 75,000, H_1 = 120) to point M_2 (Bhp_2 = 78,000, H_2 = 119.6), the new steady state for constant turbine output

FIGURE 7

OAHE POWER PLANT
SURGE TANK ANALYSIS
FULL LOAD DEMAND
TURBINE UNIT NO. 6

FIGURE 8

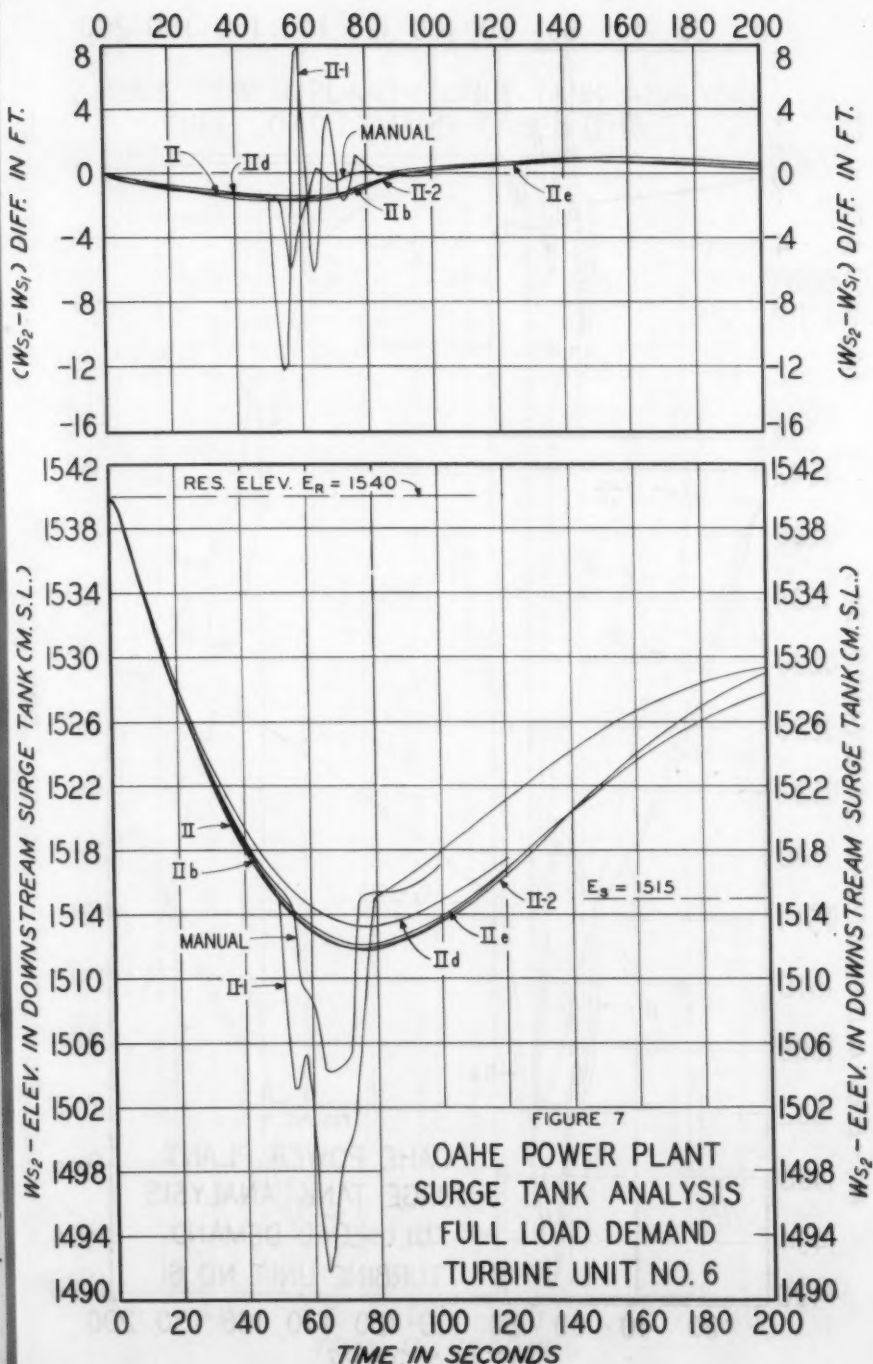OAHE POWER PLANT
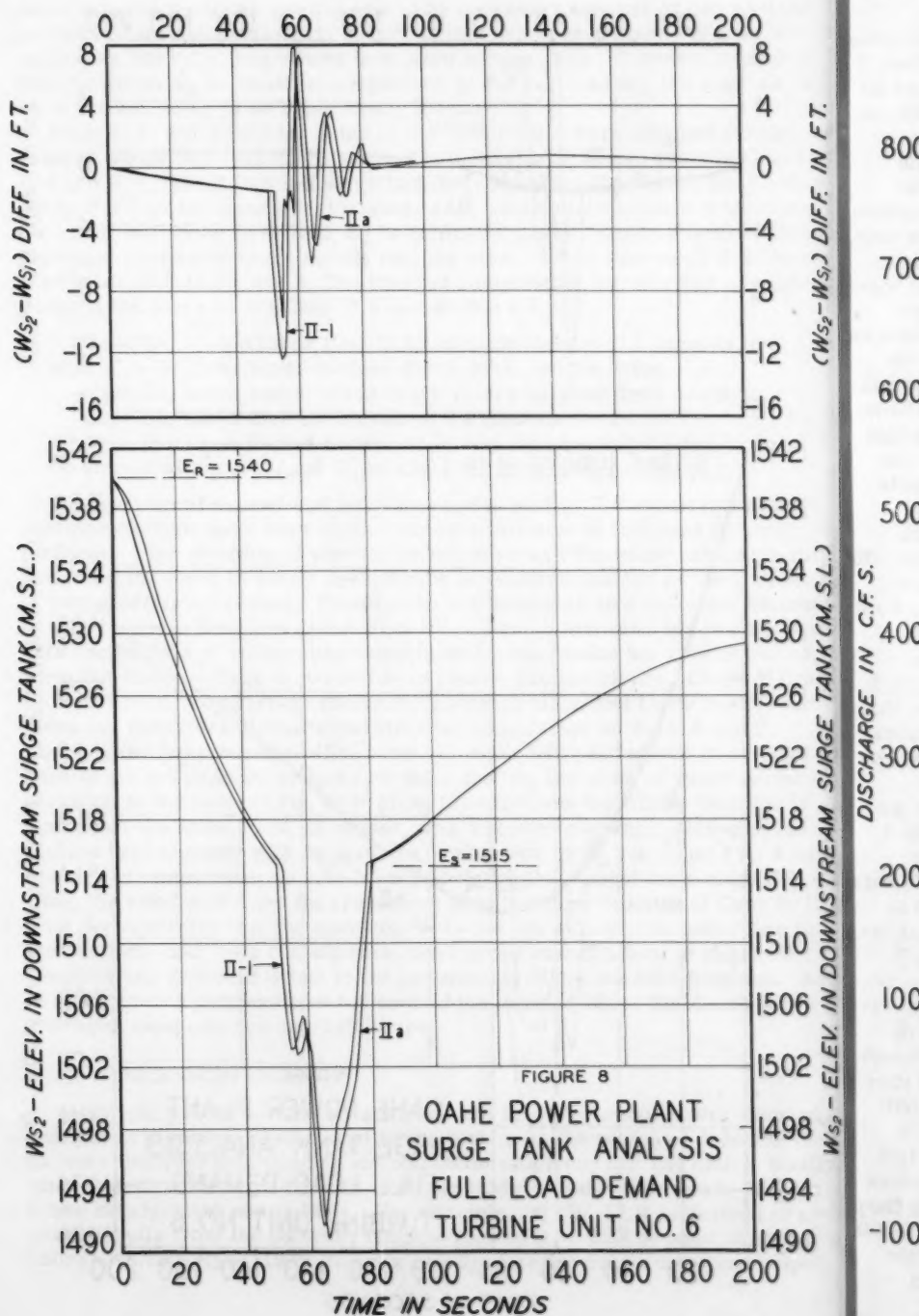SURGE TANK ANALYSIS
FULL LOAD DEMAND
TURBINE UNIT NO. 6
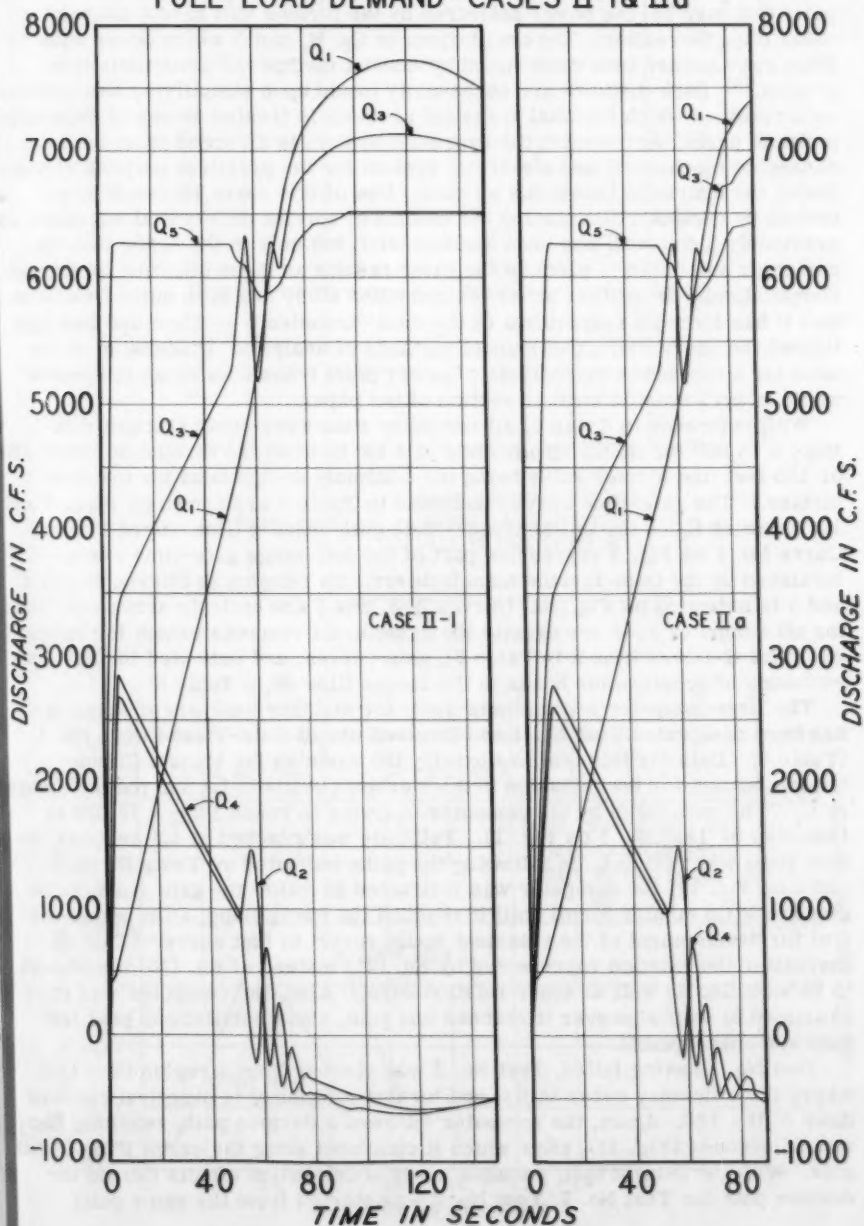
FIGURE 9

OAHE POWER PLANT-SURGE TANK ANALYSIS
FULL LOAD DEMAND-CASES II-I & IIa

CASE II-I
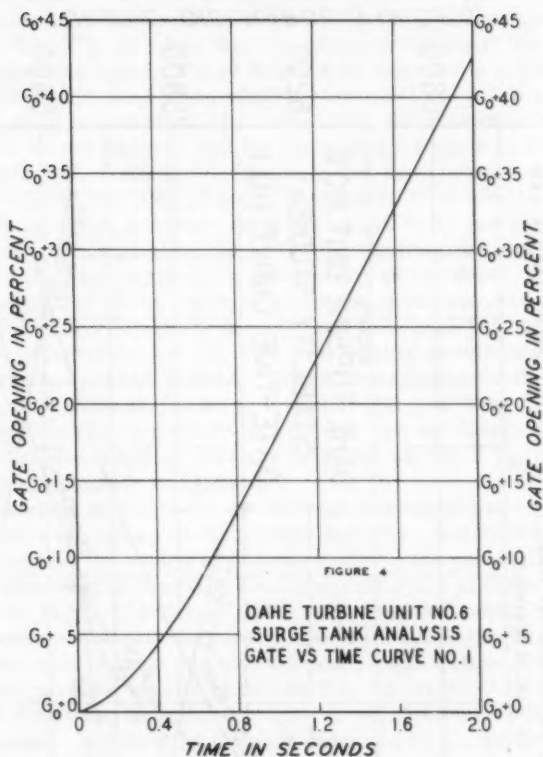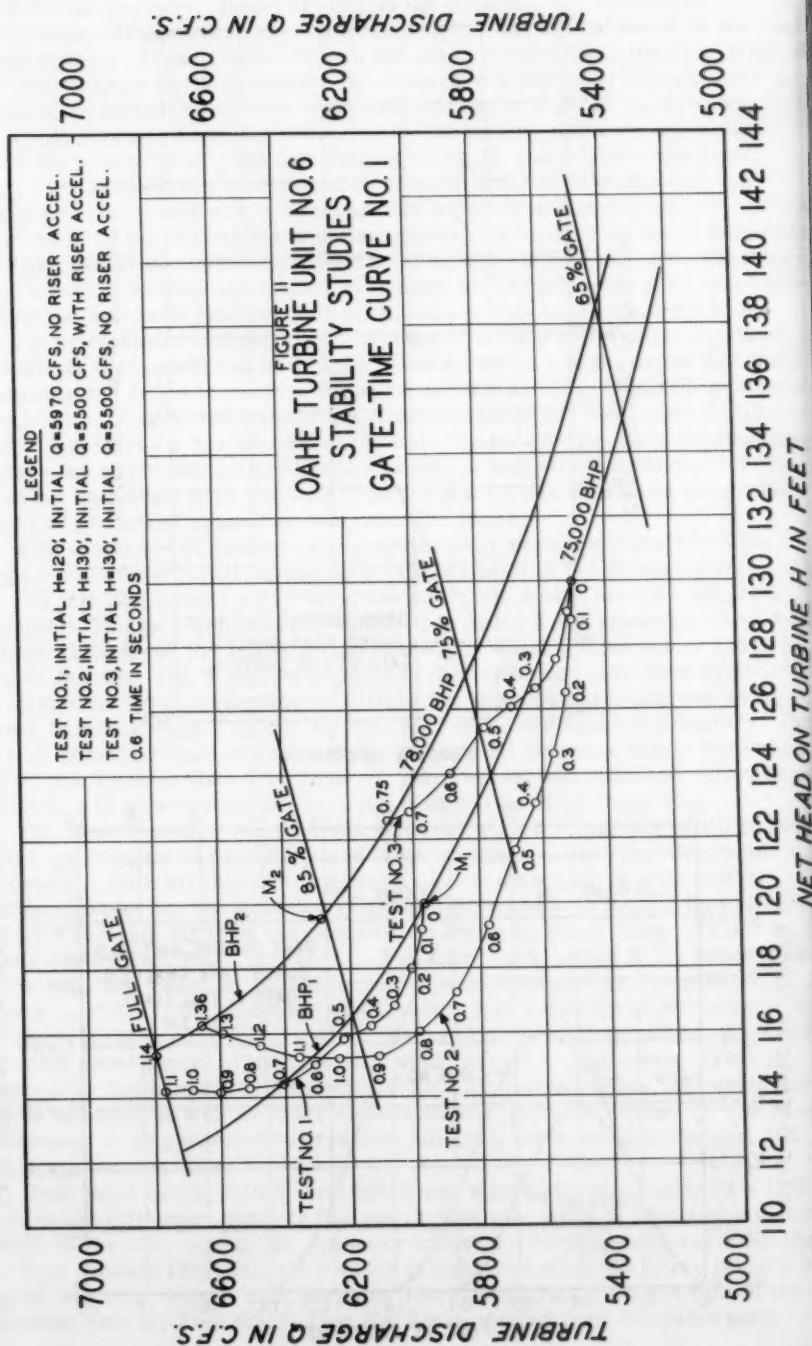
CASE II a

Left axis label: DISCHARGE IN C.F.S.

Right axis label: DISCHARGE IN C.F.S.

Far left axis labels: $(W_{S_2} - W_{S_1})$ DIFF. IN FT. W_{S_2} — ELEV. IN DOWNSTREAM SURGE TANK (M.S.L.)

TIME IN SECONDS

of 78,000 horsepower. However, this is not possible, for sudden gate movements in response to rapid governor action set up transients in the whole surge system. These transients will die out in a reasonable length of time, if the system is properly designed. By conventional methods, it is assumed for purposes of analysis that once entry onto the curve Bhp_2 is made, at whatever point that may be, the power delivered by the turbine will have a constant value Bhp_2 thereafter. The oscillations in Q_5, H, and G which occur with Bhp_2 constant are then determined by manual methods of arithmetic integration.[2] Such methods are necessarily based upon simplifying assumptions as a result of which the total transient problem is treated as one of separable multiple parts. In essence, the hydraulic system is divorced from the interconnected mechanical and electrical system for the practical purpose of analyzing the hydraulic transients as such. Use of this same approach to solutions by computer (employing the method of solving differential equations as previously discussed) has been unsuccessful, but only in the sense that the computer has failed to produce the same results as those obtained by manual computations. In another sense the computer study has been quite fruitful in that it has forced a reappraisal of the total "transient" problem and has highlighted the shortcomings of manual methods of analysis. Discussion of the need for comprehensive analysis of power plant transients as an integrable whole is presented in another section of the paper.

   With reference to Table 8, all computer runs were made starting with $Bhp_1 = 75,000$ for initial steady state at a net head on the turbine of either 120 or 130 feet (the former value being the minimum design head for the Oahe turbine.) The gate-time curves indicated in Table 8 are shown on Figs. 4 and 10, on which G_0 is the initial steady-state gate opening (line one of Table 8). Curve No. 1 on Fig. 4 represents part of the full-range gate-time curve furnished by the Oahe turbine manufacturer. Its relation to Curves No, 2, 3, and 4 is indicated on Fig. 10. Curves 2, 3, and 4 are entirely arbitrary. Data for all computer runs are exactly the same in all respects except for indicated initial H values (line 2 in Table 8), gate curves, and indicated inclusion or exclusion of acceleration heads in the risers (line 20 in Table 8).

   The first computer run that was made for stability analysis of surge tanks has been designated Test No. 1 and involved use of Gate-Time Curve No. 1 (Table 8). Data for this test are exactly the same as for Manual Computations, except for the inclusion of acceleration heads in L_5 and the difference in L_1. The path taken by the computer in trying to reach $Bhp_2 = 78,000$ is indicated by Test No. 1 on Fig. 11. Full gate was reached at 1.1 seconds before Bhp_2 was reached. In following the paths indicated by Tests No. 1, 2, and 3 on Fig. 11, the computer was instructed to follow the gate-time curve given to it (in tabular form) until it reached the curve Bhp_2, after which control for development of the transient would revert to that curve. (That is, thereafter the function represented by Eq. (21) instead of Eq. (20) would have to be satisfied as well as other relationships.) Also, the computer was programmed to stop whenever it reached full gate, since oscillations past full gate are unacceptable.

   Test No. 1 having failed, Test No. 2 was started from a region (H = 130) where the efficiency curve is flat and turbine efficiency is practical constant down to H = 120. Again, the computer followed a devious path, reaching Bhp_2 at 1.36 seconds (Fig. 11), after which it continued along the curve Bhp_2 to full gate. With the thought that, perhaps, riser acceleration effects caused the devious path for Test No. 2, Test No. 3 was started from the same point
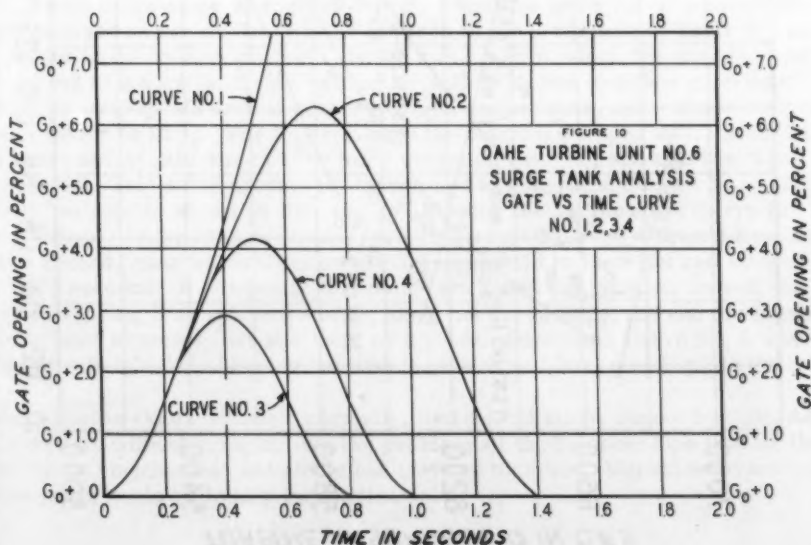
whole
e, if
for
ever

tions
rable
ter-
ana-

ns as
he
nual
l in
high-
he
ble

er 120
ne
4 and
e 8).

2, 3,
Data
dicat-
on or

tanks
o. 1
erence
is
s be-
, 2,
urve
con-
is,
have
pro-
full

130)
nstant
g Bhp₂
g to full
the
nt

FIGURE 4

OAHE TURBINE UNIT NO.6
SURGE TANK ANALYSIS
GATE VS TIME CURVE NO. 1

FIGURE 10

OAHE TURBINE UNIT NO.6
SURGE TANK ANALYSIS
GATE VS TIME CURVE
NO. 1,2,3,4

FIGURE II
OAHE TURBINE UNIT NO.6
STABILITY STUDIES
GATE-TIME CURVE NO.1

LEGEND

TEST NO.1, INITIAL H=120; INITIAL Q=5970 CFS, NO RISER ACCEL.
TEST NO.2, INITIAL H=130; INITIAL Q=5500 CFS, WITH RISER ACCEL.
TEST NO.3, INITIAL H=130; INITIAL Q=5500 CFS, NO RISER ACCEL.
0.6 TIME IN SECONDS

(H = 130) with the effects of riser acceleration excluded. This time, the path followed by the computer was satisfactory until the Bhp_2 curve was reached at 0.75 seconds (Fig. 11), but after that the computer followed the Bhp_2 curve upward to full gate once again. These failures to establish a transient (similar to that obtained by Manual Computations) that would produce oscillations in Q_5, G, and H along the constant Bhp curve of 78,000 without at any time reaching full gate do not indicate that the Oahe surge system is unstable in the region investigated. Rather, they indicate that any transient set up in the manner of conventional methods of analysis is likely to resemble the actual transient only by accident, for these methods ignore (with justification*) the oscillations of the gates with time due to other effects at the turbine and governor end of the surge system. In anticipation of any doubts that may arise concerning the adequacy of the machine program, sufficient data are given in this paper for the reader to assure himself that the computer has done the job in accordance with instructions. Machine print-out of results for Test No. 2 are given in the upper part of Table 9. These data together with the equations in Appendix III, data given in Tables 1, 5, and 8, and gate-time Curve No. 1 on Fig. 4 will permit checks on machine arithmetic in a relatively short time. It will be noted that the machine does not print out values of h_{a_2} and h_{a_4}; they may be obtained by separate computation.

When a load change is called for by external electrical power requirements, oscillations are set up in the turbine governor system itself during the first several seconds. By virtue of interconnection, the gates also oscillate. In an effort to represent at least the first one-half cycle of these oscillations, gate-time Curves No. 2, 3, and 4 (Fig. 10) were invented, using the "dead" and "cushion" ends of the manufacturer's curve (of which Curve No. 1 is a part) to form the peak regions and the tail-ends. Application of these gate-time curves then gave the results shown on Fig. 12. In all three cases the computer started and stayed within a region in which turbine efficiency is practically constant. In following the path indicated by Curve No. 2 on Fig. 12, the computer reached Bhp_2 at 0.8 second, after which it proceeded along Bhp_2 to full gate in the same manner as before. (Machine print-out of some of the results based on use of Curve No. 2 is given in the lower half of Table 9). At this point in the study a program change was made by which the machine was instructed to ignore the first intersection with Bhp_2 and to follow gate-time Curve No. 2 (Fig. 10) until it reached Bhp_2 a second time, after which control would revert to Bhp_2. For this condition the machine stopped at 1.2 seconds (near the end of gate-time Curve No. 2 shown on Fig. 10), and this test was abandoned. This same procedure was applied to gate-time Curves No. 3 and 4 with the results shown on Fig. 12. In following the path marked Curve No. 3, the computer followed instructions perfectly, reaching Bhp_2 a second time at 0.925 second, after which it followed Bhp_2 downward to the right and stopped at 1.125 seconds. It stopped at this point because it had reached the end of the table giving H as a function of Q_5 along the curve Bhp_2, the end of the table having been purposely set at a limit of H = 140. Gate-time Curve No. 4 was applied with the aim of having the path loop back and intersect Bhp_2 in the

---

*Justification exists because, normally, these oscillations cannot be described adequately without bringing into the problem all the relationships that tie the hydraulic, mechanical and electrical systems together. Manual analyses on this basis would be totally impracticable.
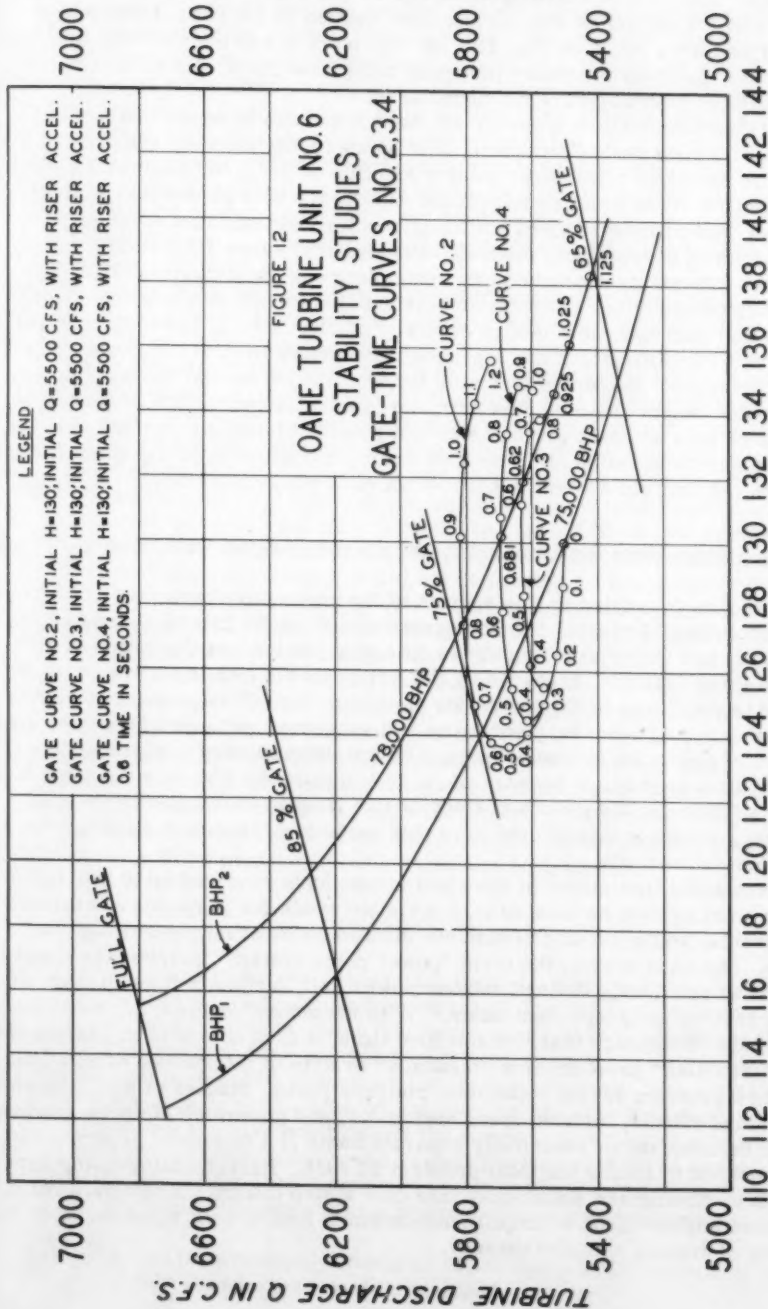
TABLE 9 OAHE POWER PLANT SURGE TANK ANALYSIS - - STABILITY STUDY - - TURBINE UNIT NO. 6

Gate Curve No. 1   Tailwater Elevation 1420.00   Reservoir Elevation E_R 1553.761

t	Q_1	h_{L_1}	h_{a_1}	P_1	$T_{1,7}$	$T_{1,2}$	Ws_1	h_{L_2}	P_8	Q_3	h_{L_3}	h_{a_3}	Q_4	Ws_2	h_{L_4}	$T_{8,9}$	$T_{8,14}$	$V^2/2g$	h_{L_5}	h_{a_5}	H	G	Q_5
0	5500	5.902	0.000		0.000	0.000	1547.86	0.000		5500	0.029	0.000	0	1547.83	0.000	0.000	0.000	2.299	0.048	0.000	130.080	68.57	5500
0.10	5500	5.902	0.585	1547.27	0.006	0.001	1547.86	0.000	1547.09	5503	0.029	0.150	3	1547.83	0.000	0.001	0.005	2.303	0.048	0.500	128.838	69.07	5505
0.20	5501	5.903	1.676	1546.18	0.026	0.004	1547.86	0.000	1545.72	5513	0.029	0.402	14	1547.83	0.001	0.004	0.029	2.321	0.049	1.410	126.558	70.07	5527
0.30	5501	5.905	2.576	1545.28	0.064	0.009	1547.86	0.003	1544.58	5532	0.030	0.610	35	1547.83	0.004	0.010	0.075	2.355	0.049	2.171	124.637	71.47	5567
0.40	5502	5.908	3.263	1544.59	0.115	0.016	1547.86	0.008	1543.68	5557	0.030	0.768	65	1547.83	0.012	0.019	0.138	2.402	0.050	2.757	123.134	72.97	5622
0.50	5504	5.911	3.916	1543.93	0.178	0.025	1547.86	0.020	1542.81	5588	0.030	0.918	101	1547.82	0.029	0.031	0.219	2.460	0.051	3.317	121.681	74.67	5689
0.60	5506	5.914	4.981	1542.87	0.257	0.036	1547.85	0.041	1541.42	5626	0.031	1.161	147	1547.82	0.061	0.045	0.322	2.532	0.053	4.213	119.362	76.87	5773
0.70	5506	5.919	5.927	1541.92	0.355	0.050	1547.85	0.077	1540.16	5672	0.031	1.374	203	1547.82	0.116	0.063	0.452	2.623	0.055	5.007	117.265	79.27	5875
0.80	5510	5.924	6.041	1541.40	0.467	0.065	1547.84	0.130	1539.41	5725	0.032	1.464	266	1547.81	0.200	0.085	0.606	2.727	0.056	5.432	116.048	81.67	5991
0.90	5513	5.929	6.730	1541.10	0.587	0.082	1547.84	0.201	1538.95	5780	0.032	1.538	333	1547.80	0.314	0.108	0.775	2.840	0.059	5.655	115.298	84.07	6113
1.00	5515	5.935	6.686	1541.14	0.710	0.100	1547.83	0.290	1538.89	5836	0.033	1.512	403	1547.79	0.457	0.134	0.954	2.957	0.061	5.588	115.240	86.47	6238
1.10	5516	5.946	6.606	1541.21	0.833	0.117	1547.82	0.391	1538.87	5890	0.033	1.474	470	1547.78	0.624	0.159	1.136	3.074	0.063	5.474	115.275	88.94	6360
1.20	5520	5.946	6.384	1541.43	0.954	0.134	1547.81	0.503	1539.04	5943	0.034	1.401	536	1547.77	0.811	0.185	1.320	3.190	0.065	5.229	115.619	91.41	6479
1.30	5523	5.951	6.140	1541.67	1.069	0.150	1547.80	0.622	1539.25	5992	0.034	1.321	599	1547.75	1.012	0.210	1.500	3.301	0.067	4.965	116.025	93.88	6591
1.36	5524	5.950	6.000	1541.61	1.138	0.159	1547.79	0.698	1539.37	6022	0.035	1.272	637	1547.74	1.143	0.226	1.611	3.369	0.068	4.786	116.269	95.43	6658
1.44	5526	5.954	6.211	1541.61	1.222	0.171	1547.78	0.795	1539.06	6057	0.035	1.297	682	1547.73	1.310	0.245	1.738	3.450	0.070	4.880	115.821	97.94	6738
1.50	5527	5.957	6.374	1541.45	1.287	0.180	1547.77	0.875	1538.81	6084	0.035	1.318	717	1547.72	1.449	0.261	1.820	3.515	0.071	4.958	115.471	100.00	6801

Gate Curve No 2   Tailwater Elevation 1420.00   Reservoir Elevation E_R 1553.761

t	Q_1	h_{L_1}	h_{a_1}	P_1	$T_{1,7}$	$T_{1,2}$	Ws_1	h_{L_2}	P_8	Q_3	h_{L_3}	h_{a_3}	Q_4	Ws_2	h_{L_4}	$T_{8,9}$	$T_{8,14}$	$V^2/2g$	h_{L_5}	h_{a_5}	H	G	Q_5
0	5500	5.902	0.000		0.000	0.000	1547.86	0.000		5500	0.029	0.000	0	1547.83	0.000	0.000	0.000	2.299	0.048	0.000	130.080	68.57	5500
0.10	5500	5.902	0.678	1547.18	0.005	0.001	1547.86	0.000	1546.98	5503	0.029	0.171	2	1547.83	0.000	0.001	0.005	2.303	0.048	0.577	128.648	69.12	5505
0.20	5501	5.903	1.704	1546.15	0.026	0.004	1547.86	0.000	1545.70	5513	0.029	0.408	14	1547.83	0.001	0.004	0.029	2.321	0.049	1.434	126.500	70.09	5527
0.30	5505	5.905	2.421	1545.43	0.062	0.009	1547.86	0.002	1544.75	5531	0.030	0.574	34	1547.83	0.003	0.010	0.073	2.354	0.049	2.042	124.958	71.32	5566
0.40	5502	5.907	2.836	1545.02	0.109	0.015	1547.86	0.008	1544.21	5554	0.030	0.670	61	1547.83	0.011	0.018	0.130	2.396	0.050	2.403	124.022	72.51	5615
0.50	5503	5.910	2.963	1544.89	0.160	0.022	1547.86	0.016	1544.00	5579	0.030	0.699	91	1547.83	0.023	0.027	0.195	2.443	0.051	2.525	123.671	73.57	5670
0.60	5505	5.912	2.870	1544.98	0.212	0.029	1547.85	0.028	1544.06	5604	0.030	0.677	121	1547.82	0.041	0.037	0.263	2.491	0.052	2.462	123.773	74.47	5725
0.70	5506	5.914	2.274	1545.57	0.260	0.036	1547.85	0.042	1544.75	5627	0.031	0.538	149	1547.82	0.062	0.046	0.326	2.535	0.053	1.979	124.923	74.87	5776
0.80	5506	5.916	1.039	1546.81	0.290	0.041	1547.85	0.052	1546.23	5642	0.031	0.252	167	1547.82	0.079	0.053	0.370	2.564	0.053	0.964	127.411	74.47	5809
0.81	5506	5.916	0.948	1546.90	0.292	0.041	1547.85	0.052	1546.34	5642	0.031	0.231	168	1547.81	0.080	0.052	0.370	2.56	0.053	0.888	127.597	74.42	5810

FIGURE 12

OAHE TURBINE UNIT NO.6
STABILITY STUDIES
GATE-TIME CURVES NO.2,3,4.

LEGEND

GATE CURVE NO.2, INITIAL Q=5500 CFS, WITH RISER ACCEL.
GATE CURVE NO.3, INITIAL Q=5500 CFS, WITH RISER ACCEL.
GATE CURVE NO.4, INITIAL Q=5500 CFS, WITH RISER ACCEL.
0.6 TIME IN SECONDS.

opposite direction from that which had been indicated by Curve No. 3. The computer started a return loop along Curve No. 4 as indicated by the points at 0.9 and 1.0 second on Fig. 12 and then stopped at 1.0 second (the end of gate-time Curve No. 4 on Fig. 10). At this point it was decided that there was altogether too much arbitrary judgment being exercised, and further computer studies of stability were not undertaken.

Machine-computation time for all these tests was comparatively small. However, a good deal of time was spent in analyzing machine results and in trying to establish a transient using a function of the form given by Eq. (21) as a control. It is concluded from the experience thus gained that analyses utilizing this procedure yield artificial results and that, as a consequence, the results of conventional methods of analysis of surge-tank stability along the lines previously described are incomplete and inconclusive. This does not raise or imply any reflection on the excellent work that has been done in connection with hydraulic design of power plants. For, it has long been recognized that simplifications are a practical necessity in order to render power plant "transient" problems tractable for solution by manual methods in a reasonable length of time. However, successful adaptation for computer solutions of very complex problems in many fields of engineering and science now raises the question as to the necessity for continuing to use simplified procedures that yield results that are not entirely satisfactory.

Some Ideas for Comprehensive Study by Digital Computer

The ideas presented in this section of the paper have been extracted from an unpublished "Proposal for Comprehensive Study by Digital Computer of Hydraulic and Governing Transients Affecting Design and Operation of a Power Plant System", prepared by the writer on his own initiative and submitted to the Corps of Engineers for consideration. This proposal includes the derivation of necessary equations and suggested methods of machine analysis and computation for undertaking a comprehensive study, which is only briefly discussed below because of space limitations. The only reason for inviting attention to the proposed study at this time is to indicate to the engineering profession that efforts are being made to overcome the limitations of conventional methods.

Formidable limitations of time and means have required up to now that a power plant system be treated as a divisible whole for purposes of analysis of hydraulic and governing transients induced by external power-load variations. (As used herein, the term "power plant system" includes the surge system as previously defined and one power unit, turbine and generator, supplying load on an independent basis.) With the advent of powerful, high-speed computers, it appears that, for the first time, it is practicable to analyze the total "transient" problem as a "feedback" system of interwoven effects rather than as a problem having separable, multiple parts. Studies of water hammer, surge-tank effects, turbine-speed regulation, and governing stability need no longer be made on an essentially separate basis if a computer program can be developed to handle the total problem as such. Recent analog-computer studies by Koenig and Knudtson[9] are only a step toward the ultimate goal, for these studies involve simplifications which tend to cast some doubt on the validity of results obtained thereby.

It appears that studies of the "total problem" can be made with conclusive results only when the relationships controlling hydraulic behavior in the conduit, penstock, and surge-tank system are solved simultaneously with the connecting relationships between turbine output, the governor, machine inertia and speed, and external power load. To do so requires the simultaneous solution of a great number of equations, short time step by short time step. The time step would probably have to be in the order of hundredths of a second (or less in some cases) in order to account for all pressure disturbances and reflections* generated during the existence of a transient. Obviously, such an approach is totally impractical, if not impossible, for manual computation, but, apparently, not for a modern digital computer having the capability and computational speed comparable with those of an IBM 704 computer.

It has been roughly estimated by the writer that the machine time required per total solution on such a computer would be well within practical range, if the necessary machine program can be successfully developed. A total solution would yield time histories of velocity and piezometric heads at a large number of pre-determined points within the surge system as well as time histories of turbine power output, turbine discharge and head, turbine and generator speed for any assumed or given external power-load variation with time applied at the power unit under study. Important quantities such as surge tank size, orifice characteristics, penstock and conduit characteristics, machine inertia, governor characteristics, etc., would all be introduced in the total problem as parameters. By the proposed methods, water-hammer and mass-oscillation phenomena would not be treated as separate entities. Aside from the basic assumption of essentially independent operation of the power unit, no other simplifications of important significance appear to be necessary for the methods of solution that have been proposed. To accomplish all this on a digital computer requires the simultaneous solution by numerical methods of partial-differential equations within the surge system and of ordinary differential equations at the turbine end of the system. Methods for doing this are apparently available. It remains to be determined whether or not the fundamental partial-differential equations describing hydraulic behavior within the surge system can be solved by computer on a practical basis. The writer believes that this can be done, but a machine programming job of great scope will have to be performed before success is achieved.

CONCLUSIONS

A machine program for the IBM 650 computer is now available for solving load-rejection and full-load demand problems of a surge system comparable to that at the Oahe project in one to two hours of machine-computation time per case. The machine time per solution for installations having only one riser connected to the main conduit will be materially less. The program described herein may be easily adapted for such cases. The relatively small time required per machine solution and the comparatively small costs involved make it feasible to enlarge the scope of design studies of surge systems well beyond the limits ordinarily imposed by manual methods of solution. By examining the results of a number of machine solutions readily

---

*In the study that is envisioned, effects of the compressibility of water and the elasticity of flow boundaries would be included.

obtained as a result of changing one or two major factors from one machine run to another, a better understanding of problem phenomena for a particular installation will be obtained. As a consequence, construction savings may be achieved in some cases.

A digital computer program which will yield solutions for transient effects resulting from small-load changes from one steady-state power output to another is not yet available despite considerable efforts to develop such a program. It may be entirely possible to develop such a program by utilizing methods of solution other than those described herein. However, it appears that any method which ignores the applicable relationships between turbine, governor, machine speed and inertia, and external power load for the purpose of analyzing the hydraulic transients as such is bound to yield inconclusive results. The premise that such simplifications give conservative results insofar as surge-tank stability is concerned may be true but the degree of conservatism is not known in application to particular cases. For a long time a need has existed for analyzing the total "transient" problem without the major simplifications that are ordinarily made. When only manual methods of analysis and computation were available, fulfillment of this need on a practical basis was not possible. With the advent of powerful, high-speed computers the prospects are now excellent that this need can be wholly satisfied. Some efforts have already been made toward the ultimate goal of adapting the total "transient" problem of a power plant for solution by a digital computer in such a way that machine results for water hammer, surge-tank effects, turbine-speed regulation, and governing stability would be delivered in one package in a few hours of machine-computation time. If successful adaptation is ultimately achieved, many benefits and possible construction savings will accrue.

ACKNOWLEDGMENTS

APPENDIX I

Notation

A - Cross-sectional area in sq. ft. of conduit and penstock.

A_o - Area of orifice in sq. ft.

A_r - Area of riser in sq. ft.

A_f - Area of surge tank in sq. ft.

Bhp   - Brake horsepower (output of turbine)

C_o - Coefficient of orifice in $Q = C_o A_o \sqrt{2gh_o}$

C_r - Coefficient for risers, $\dfrac{1}{2ga_r^2}$ or $\dfrac{1+K_e'}{2gA_r^2}$

C_D, C_D' - Coefficients of loss for diverging flow in tees, function of discharge ratio $(Q_{riser}/Q_{conduit})$

C_C, C_C' - Coefficients of loss for converging flow in tees, function of discharge ratio $(Q_{riser}/Q_{conduit})$

C_t - General coefficient of loss for the tees in $T = C_t \dfrac{V^2}{2g}$

D   - Diameter in feet of conduit and penstock.

D_r - Diameter of riser in feet.

D_o - Diameter of orifice in feet.

D_f - Diameter of surge tank in feet.

e_c - Absolute roughness length in feet for concrete.

e_s - Absolute roughness length in feet for steel.

ϵ - Pre-assigned margin of relative error.

E_R - Elevation of reservoir in feet above M.S.L.

E_s - Elevation of base of surge tanks in feet above M.S.L.

E_t - Elevation of centerline of turbine distributor.

f   - Darcy-Weisbach friction factor, function of Reynolds number.

f_c, f_s - Friction factors in upstream conduit.

f_2, f_4 - Friction factors in upstream and downstream risers, respectively.

f_3 - Friction factor in penstock between risers (Length L_3).

f_5 - Friction factor in penstock (Length L_5).

F_c, F_s - Functions of Reynolds Number for concrete and steel.

F, F_g, F_q - Functions defined by Eq. (III-46), (III-45), and (III-49).

g   - Acceleration due to gravity in ft./sec.^{2}

G   - Gate opening in per cent for turbine wicket gates.

G_o - Initial gate opening for steady state.

h_f - Surface resistance loss in feet.

h_s - Form loss in feet.

h_L - Head loss in feet (may include all types of losses and velocity head).

h_{L_1} - Head loss in feet in upstream conduit in length L_1 (includes all form and surface-resistance losses as well as velocity head), defined by Eq. (III-21).

h_{L_2}, h_{L_4} - Head losses in upstream and downstream risers, respectively, defined by Eqs. (III-26) and (III-29).

h_{L_3} - Head loss in penstock L_3, defined by Eq. (III-24).

h_{L_5} - Head loss in penstock length L_5, defined by Eq. (III-25).

H   - Net head on turbine in feet.

h_0 - Head on orifice in feet (piezometric drop).

h_a - Acceleration head in feet, defined as $\dfrac{L}{gA}\dfrac{dQ}{dt}$

h_{a_1} - Acceleration head in upstream conduit (L_1).

h_{a_2}, h_{a_4} - Acceleration heads in upstream and downstream risers, respectively.

h_{a_3} - Acceleration head in penstock length L_3.

h_{a_5} - Acceleration head in penstock length L_5.

h_v - Velocity head in feet.

K   - A constant, in general.

K_1 - A constant defined by Eq. (III-22).

K_1' - A constant, $\dfrac{1}{2gDA^2}$

K_2, K_4 - Constants which apply to upstream and downstream risers, respectively, defined by Eqs. (III-28) and (III-31).

K_e - Entrance loss coefficient for upstream conduit.

K_e' - Entrance loss coefficient, surge tank to riser.

K_{tr} - Loss coefficient for trashracks.

K_s - Loss coefficient for all gate slots.

K_{ts} - Loss coefficient for all transitions.

K_{hb} - Loss coefficient for all horizontal bends.

K_{vb} - Loss coefficient for all vertical bends.

K_{ot} - Coefficient to account for all other form losses.

K_v - Velocity-head coefficient = 1.0.

L_1, L_3, L_5 - Lengths in feet in main conduit (see Fig. 1).

L_c - Portion of L_1 lined with concrete.

L_s - Portion of L_1 lined with steel.

N_R - Reynolds Number, $\dfrac{VD}{\nu}$

P	- Piezometric head in feet above M.S.L.
$P_1, P_2, P_3 \ldots$	- Piezometric heads at indicated points (see Fig. 1).
Q_1	- Discharge in c.f.s. in upstream conduit.
Q_2, Q_4	- Discharges in c.f.s. in upstream and downstream risers, respectively.
Q_3	- Discharge in c.f.s. in penstock length L_3 (Fig. 1).
Q_5	- Discharge in c.f.s. in penstock length L_5 (Fig. 1).
$\lvert Q \rvert$	- Indicates absolute value of Q (ignore sign).
R	- Height of riser in feet (Fig. 1).
R_y	- Relative error of y variables.
r_1, r_3, r_5	- Constants defined by Eqs. (III-1), (III-2), and (III-3).
r_2, r_4	- Constants defined by Eqs. (III-4) and (III-5).
t	- Time in seconds.
Δt	- Time interval.
T	- Tee loss in feet.
$T_{1,2}$	- Tee loss in feet between points 1 and 2 (Fig. 1).
$T_{1,7}, T_{8,9}, T_{8,14}$	- Tee losses in feet between indicated points.
Tw	- Tailwater elevation in feet above M.S.L.
V_1	- Velocity in ft./sec. in length L_1.
V_2, V_4	- Velocities in ft./sec. in upstream and downstream risers, respectively.
V_3	- Velocity in ft./sec. in length L_3.
V_5	- Velocity in ft./sec. in length L_5.
y	- Used to represent any one of variables, Q_1, Q_3, Q_5, Ws_1, and Ws_2.
Ws_1	- Water surface elevation (M.S.L.) of upstream surge tank.
Ws_2	- Water surface elevation (M.S.L.) of downstream surge tank.
ν	- Kinematic viscosity of water in feet squared per second.
α, β	- Functions defined by Eqs. (III-15) and (III-16).
θ, ψ	- Functions defined by Eqs. (III-18) and (III-19).

APPENDIX II

REFERENCES

1. Jaeger, C., "Engineering Fluid Mechanics", Blackie &Son, Ltd., London, 1956.

2. Rich, G. R., "Hydraulic Transients", Engineering Societies Monograph, McGraw-Hill Book Co., 1951.

3. Rouse, H., "Engineering Hydraulics", John Wiley & Sons, Inc., 1950.

4. Vogel, G., "Experiments To Determine The Loss In The Right Angle Pipe Tees", Translation from German, Tech. Memo. 299, U. S. Bureau of Reclamation, August 1932.

5. Johnson, W. E., "Missouri River Basin Plan In Operation", Proc. Paper 800, ASCE, Sept. 1955.

6. Pafford, R. J., Jr., "Operation Of Missouri River Main Stem Reservoirs", Proc. Paper 1370, ASCE, Sept. 1957.

7. Bradley, J. N. and Thompson, L. R., "Friction Factors For Large Conduits Flowing Full", U. S. Bureau of Reclamation, Eng. Monographs No. 7, March 1951.

8. Paynter, H. M., "Surge And Water Hammer Problems", Symposium, Transactions ASCE, Vol. 118, page 962, 1953.

9. Koenig, E. C. and Knudtson, H. A., "Computer Studies of Penstock and Governor Systems", Proc. Paper 1489, ASCE, December 1957.

10. Fritz, W. B. and Moraff, N., "Automatic Error Control In The Solution Of Differential Equations On The IBM 650", paper presented at ACM Meeting at U. of Houston, June 19, 1957.

APPENDIX III

Mathematical Relationships

Notation

Detailed notation is given in Appendix I. In general, the following notation is utilized, based upon the pound-foot-second system:

A	- Cross-sectional area	H	- Net head on turbine
C	- Coefficient (riser, orifice, tee)	K	- Coefficient for losses
D	- Diameter	L	- Length in conduit
E	- Elevation (M.S.L.)	N_R	- Reynolds Number
e	- Absolute roughness length	P	- Piezometric head (M.S.L.)
F	- Function	Q	- Discharge
f	- Friction factor	R	- Height of riser
G	- Gate opening in per cent	T	- Tee loss

h_a - Acceleration head   t   - Time

h_L - Lost head   V   - Velocity

h_v - Velocity head   Ws   - Water surface elevation

Sign Convention

The net head on the turbine and all piezometric heads are always positive. The signs of all lost heads and acceleration heads are automatically assigned by the equations that define them. The signs of all velocities will be the same as those of the corresponding discharges, since $Q = AV$. The sign convention for all discharges is as follows:

All flows Q directed <u>toward</u> the turbine are given a positive (+) sign.
All flows Q directed <u>away</u> from the turbine are given a <u>negative</u> (-) sign.
Flows Q in the risers are positive when directed <u>downward</u> and <u>negative</u> when directed upward into the surge tanks.

Mathematical Relationships

The physical equations and functions required to solve surge problems for the system shown in Fig. 1 are given in this appendix. The major variables which vary with time t are: all Q's, all V's, all P's, all f's, Ws_1, Ws_2, G, H, C_D, C'_D, C_c, and C'_c. Such quantities as A_f, R, E_s, D, C_0, etc., may enter the machine program as parameters; that is, such quantities are constant during any one run, but may be varied from one run to another.

$$\frac{dQ_1}{dt} = gr_1(E_R - P_1 - h_{L_1}) \qquad\qquad r_1 = \frac{A}{L_1} \qquad\qquad \text{(III-1)}$$

$$\frac{dQ_3}{dt} = gr_3(P_7 - P_8 - h_{L_3}) \qquad\qquad r_3 = \frac{A}{L_3} \qquad\qquad \text{(III-2)}$$

$$\frac{dQ_5}{dt} = gr_5(P_{14} - P_{15} - h_{L_5}) \qquad\qquad r_5 = \frac{A}{L_5} \qquad\qquad \text{(III-3)}$$

$$\frac{dQ_2}{dt} = gr_2(Ws_1 - P_2 - h_{L_2}) \quad\begin{cases} r_2 = \dfrac{A_r}{R}, & Ws_1 \geq E_s \\[2mm] r_2 = \dfrac{A_r}{Ws_1 - E_s + R}, & Ws_1 < E_s \end{cases} \qquad \text{(III-4)}$$

$$\frac{dQ_4}{dt} = gr_4(Ws_2 - P_9 - h_{L_4}) \quad\begin{cases} r_4 = \dfrac{A_r}{R}, & Ws_2 \geq E_s \\[2mm] r_4 = \dfrac{A_r}{Ws_2 - E_s + R}, & Ws_2 < E_s \end{cases} \qquad \text{(III-5)}$$

$$Q_1 + Q_2 = Q_3 \qquad\qquad\qquad \text{(III-6)}$$

$$Q_3 + Q_4 = Q_5 \text{ , or } Q_1 + Q_2 + Q_4 = Q_5 \qquad\qquad \text{(III-7)}$$

$$\frac{dQ_1}{dt} + \frac{dQ_2}{dt} = \frac{dQ_3}{dt} \quad \text{(by differentiation of III-6)} \tag{III-8}$$

$$\frac{dQ_3}{dt} + \frac{dQ_4}{dt} = \frac{dQ_5}{dt}, \text{ or } \frac{dQ_1}{dt} + \frac{dQ_2}{dt} + \frac{dQ_4}{dt} = \frac{dQ_5}{dt} \tag{III-9}$$

Substituting from (III-1), (III-4), and (III-2), Eq. (III-8) becomes

$$r_1(E_R - P_1 - h_{L_1}) + r_2(Ws_1 - P_2 - h_{L_2}) = r_3(P_7 - P_8 - h_{L_3}) \tag{III-10}$$

Substituting from (III-1), (III-4), III-5), and (III-3), Eq. (III-9) becomes

$$r_1(E_R - P_1 - h_{L_1}) + r_2(Ws_1 - P_2 - h_{L_2}) + r_4(Ws_2 - P_9 - h_{L_4}) = r_5(P_{14} - P_{15} - h_{L_5}) \tag{III-11}$$

$$P_1 - P_2 = T_{1,2} \quad \text{and} \quad P_1 - P_7 = T_{1,7} \tag{III-12}$$

$$P_8 - P_9 = T_{8,9} \quad \text{and} \quad P_8 - P_{14} = T_{8,14} \tag{III-13}$$

Using (III-12), eliminate P_2 and P_7 from (III-10) to obtain

$$P_8 = \alpha\, P_1 + \beta, \text{ in which} \tag{III-14}$$

$$\alpha = \frac{r_1 + r_2 + r_3}{r_3}, \text{ and} \tag{III-15}$$

$$\beta = -T_{1,7} - h_{L_3} - \frac{r_1}{r_3}(E_R - h_{L_1}) - \frac{r_2}{r_3}(Ws_1 + T_{1,2} - h_{L_2}). \tag{III-16}$$

Using (III-12) and (III-13), eliminate P_2, P_9, P_{14} from (III-11) to obtain

$$P_8 = \theta\, P_1 + \gamma, \text{ in which} \tag{III-17}$$

$$\theta = \frac{-r_1 - r_2}{r_4 + r_5}, \text{ and} \tag{III-18}$$

$$\gamma = \frac{r_1}{r_4 + r_5}(E_R - h_{L_1}) + \frac{r_2}{r_4 + r_5}(Ws_1 + T_{1,2} - h_{L_2}) \tag{III-19}$$

$$+ \frac{r_4}{r_4 + r_5}(Ws_2 + T_{8,9} - h_{L_4}) + \frac{r_5}{r_4 + r_5}(P_{15} + h_{L_5} + T_{8,14})$$

Solving (III-14) and (III-17) simultaneously yields

$$P_1 = \frac{\gamma - \beta}{\alpha - \theta} \tag{III-20}$$

Knowing P_1 from (III-20), P_8 may be obtained from either (III-14) or 17. P_2, P_7, P_9, and P_{14} may then be evaluated by use of (III-12) and (III-13). To evaluate (III-20), it is first necessary to determine the head losses h_L and tee losses T in (III-16) and (III-19).

Head Losses are determined by Eqs. (III-21) through (III-31), as follows:

$$h_{L_1} = \left[K_1 + K_1'(f_c L_c + f_s L_s)\right] |q_1| q_1 \tag{III-21}$$

$$K_1 = \frac{1 + \Sigma K}{2gA^2}, \text{ with } \Sigma K = K_{tr} + K_e + K_s + K_{ts} + K_{hb} + K_{vb} + K_{ot} \tag{III-22}$$

$$K_1' = \frac{1}{2gDA^2}$$

$$f_c = F_c(N_R) = F_c\left(\frac{VD}{\nu}\right) , \quad \frac{e_c}{D} = \text{constant} \tag{III-23}$$

$$f_s = F_s(N_R) = F_s\left(\frac{VD}{\nu}\right) , \quad \frac{e_s}{D} = \text{constant}$$

Tables of values for f_c and f_s are needed for machine runs; see, for example, Tables 1, 2, and 4 herewith. Similarly, for all other friction factors, f_2, f_3, f_4, f_5, all of which are functions of N_R, and same tables may be used.

$$h_{L_3} = K_1' f_3 L_3 \left|Q_3\right| Q_3 \tag{III-24}$$

$$h_{L_5} = K_1' f_5 L_5 \left|Q_5\right| Q_5 \tag{III-25}$$

$$\tag{III-26}$$

$$h_{L_2} = \left(C_r + K_2 f_2 + \frac{1}{2gC_o^2 A_o^2}\right) \left|Q_2\right| Q_2$$

$$C_r = \frac{1}{2gA_r^2} , \quad \text{when } Q_2 \text{ is negative and } Ws_1 \geq E_s. \tag{III-27}$$

$$C_r = \frac{1 + K_e'}{2gA_r^2} , \quad \text{when } Q_2 \text{ is positive and } Ws_1 > E_s.$$

$$C_r = 0, \quad \text{when } Q_2 \text{ is positive and } Ws_1 = E_s.$$

$$C_r = 0, \quad \text{when } Ws_1 < E_s, \text{ irrespective of sign of } Q_2.$$

$$K_2 = \frac{R}{2gD_r A_r^2} , \quad \text{when } Ws_1 \geq E_s. \tag{III-28}$$

$$K_2 = \frac{Ws_1 - E_s + R}{2gD_r A_r^2}, \quad \text{when } Ws_1 < E_s.$$

$$h_{L_4} = \left(C_r + K_4 f_4 + \frac{1}{2gC_o^2 A_o^2}\right) \left|Q_4\right| Q_4 \tag{III-29}$$

$$\tag{III-30}$$

$$C_r = \frac{1}{2gA_r^2} , \quad \text{when } Q_4 \text{ is negative and } Ws_2 \geq E_s.$$

$$C_r = \frac{1 + K_e'}{2gA_r^2} , \quad \text{when } Q_4 \text{ is positive and } Ws_2 > E_s.$$

$$C_r = 0, \quad \text{when } Q_4 \text{ is positive and } Ws_2 = E_s.$$

$$C_r = 0, \quad \text{when } Ws_2 < E_s, \text{ irrespective of sign of } Q_4.$$

$$K_4 = \frac{R}{2gD_r A_r^2} , \quad \text{when } Ws_2 \geq E_s. \tag{III-31}$$

$$K_4 = \frac{Ws_2 - E_s + R}{2gD_r A_r^2} , \quad \text{when } Ws_2 < E_s.$$

Tee Losses are evaluated in accordance with the following summarization Cases A through E (Eqs. (III-32) through (III-39)). The coefficients C_D, C_D', C_c, and C_c' are evaluated from tables; see, for example, Table 5 herewith. For ease in writing, let $h_{v1} = \dfrac{V_1^2}{2g}$, $h_{v3} = \dfrac{V_3^2}{2g}$, and $h_{v5} = \dfrac{V_5^2}{2g}$, and recall that $T_{1,2} = P_1 - P_2$, $T_{1,7} = P_1 - P_7$, etc., as indicated by Eqs. (III-12) and (III-13).

Case A - Tee losses ignored.

$$P_1 = P_2 = P_7 \text{ and } P_8 = P_9 = P_{14}$$

Case B - Q_1 and Q_3 are both positive and Q_2 is negative.
          Q_3 and Q_5 are both positive and Q_4 is negative.

$$T_{1,2} = C_D \, h_{v_1} \qquad\qquad T_{8,9} = C_D \, h_{v_3} \qquad (III-32)$$

$$T_{1,7} = C_D' \, h_{v_1} \qquad\qquad T_{8,14} = C_D' \, h_{v_3} \qquad (III-33)$$

C_D and C_D' = function $\left|\dfrac{Q_2}{Q_1}\right|$ $\qquad$ C_D and C_D' = function $\left|\dfrac{Q_4}{Q_3}\right|$ $\quad$ Table

Case C - Q_1 and Q_3 are negative and Q_2 is negative.
          Q_3 and Q_5 are negative and Q_4 is negative.

$$T_{1,2} = (C_D - C_D') \, h_{v_3} \qquad\qquad T_{8,9} = (C_D - C_D') \, h_{v_5} \qquad (III-34)$$

$$T_{1,7} = -C_D' \, h_{v_3} \qquad\qquad T_{8,14} = -C_D \, h_{v_5} \qquad (III-35)$$

C_D and C_D' = function $\left|\dfrac{Q_2}{Q_3}\right|$ $\qquad$ C_D and C_D' = function $\left|\dfrac{Q_4}{Q_5}\right|$ $\quad$ Table

Case D - Q_1 and Q_3 are positive and Q_2 is positive.
          Q_3 and Q_5 are positive and Q_4 is positive.

$$T_{1,7} = C_c' \, h_{v_3} \qquad\qquad T_{8,14} = C_c' \, h_{v_5} \qquad (III-36)$$

$$T_{1,2} = (C_c' - C_c) h_{v_3} \qquad\qquad T_{8,9} = (C_c' - C_c) \, h_{v_5} \qquad (III-37)$$

C_c and C_c' = function $\left|\dfrac{Q_2}{Q_3}\right|$ $\qquad$ C_c and C_c' = function $\left|\dfrac{Q_4}{Q_5}\right|$ $\quad$ Table

Case E - Q_1 and Q_3 are negative and Q_2 is positive.
          Q_3 and Q_5 are negative and Q_4 is positive.

$$T_{1,2} = -C_c \, h_{v_1} \qquad\qquad T_{8,9} = -C_c \, h_{v_3} \qquad (III-38)$$

$$T_{1,7} = -C_c' \, h_{v_1} \qquad\qquad T_{8,14} = -C_c' \, h_{v_3} \qquad (III-39)$$

C_c and C_c' = function $\left|\dfrac{Q_2}{Q_1}\right|$ $\qquad$ C_c and C_c' = function $\left|\dfrac{Q_4}{Q_3}\right|$ $\quad$ Table

As long as the form of the equations and the method of evaluation of tee losses do not change, any set of numerical values may be used for the coefficients in the machine program.

<u>Acceleration heads</u> are defined as follows:

$$h_{a_1} = \frac{L_1}{gA} \frac{dQ_1}{dt} ; \quad h_{a_3} = \frac{L_3}{gA} \frac{dQ_3}{dt} ; \quad h_{a_5} = \frac{L_5}{gA} \frac{dQ_5}{dt} \tag{III-40}$$

$$h_{a_2} = \frac{R_2}{gA_r} \frac{dQ_2}{dt} \quad \begin{cases} R_2 = R, \text{ when } Ws_1 \geq E_s. \\[2mm] R_2 = Ws_1 - E_s + R, \text{ when } Ws_1 < E_s. \end{cases} \tag{III-41}$$

$$h_{a_4} = \frac{R_4}{gA_r} \frac{dQ_4}{dt} \quad \begin{cases} R_4 = R, \text{ when } Ws_2 \geq E_s. \\[2mm] R_4 = Ws_2 - E_s + R, \text{ when } Ws_2 < E_s. \end{cases} \tag{III-42}$$

<u>Water surfaces</u> in surge tanks depend upon following equations as well as others:

$$\frac{d(Ws_1)}{dt} = -\frac{Q_2}{A_{f_2}} \quad \begin{cases} A_{f_2} = A_f, \text{ when } Ws_1 > E_s. \\[2mm] A_{f_2} = A_f, \text{ when } Ws_1 = E_s \text{ and } Q_2 \text{ is } -. \\[2mm] A_{f_2} = A_r, \text{ when } Ws_1 = E_s \text{ and } Q_2 \text{ is } +. \\[2mm] A_{f_2} = A_r, \text{ when } Ws_1 < E_s. \end{cases} \tag{III-43}$$

$$\frac{d(Ws_2)}{dt} = -\frac{Q_4}{A_{f_4}} \quad \begin{cases} A_{f_4} = A_f, \text{ when } Ws_2 > E_s. \\[2mm] A_{f_{1F}} = A_f, \text{ when } Ws_2 = E_s \text{ and } Q_4 \text{ is } -. \\[2mm] A_{f_4} = A_r, \text{ when } Ws_2 = E_s \text{ and } Q_4 \text{ is } +. \\[2mm] A_{f_4} + A_r, \text{ when } Ws_2 < E_s. \end{cases} \tag{III-44}$$

At <u>Turbine</u> end of the surge system; the following relationships must also be satisfied:

$$G = F_g(t) \tag{III-45}$$

Gives the gate opening as a function of time. Either given by manufacturer or must be assumed. The function enters the machine program as a table; hence, it may have any form.

$$H = F(G, Q_5) \tag{III-46}$$

Turbine-performance curves enter the machine program as a table; see for example, Table 3 herewith. In program, G and Q_5 are the independent

variables and H is the dependent variable; that is, H is computed when G and Q_5 are known.

$$H = P_{15} + \frac{V_5^2}{2g} - Tw, \text{ in which} \tag{III-47}$$

$$P_{15} = P_{14} - h_{L_5} - h_{a_5}. \tag{III-48}$$

$$H = F_q (Q)_{Bhp} = \text{constant} \tag{III-49}$$

This function used <u>only</u> for <u>stability</u> studies; see, for example, Figs. 11 and 12 herewith. These curves enter the program as tables.

Proc

Joe M

Two m
Reclamat
for the de
amples o

The d
Bureau o
to trace
design of
ing basin
informat
backwate
easemen
cilities
located a

The r
referred
profiles
all segm
pecially
main ch
on a pro
sectiona
also be

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

TWO METHODS TO COMPUTE WATER SURFACE PROFILES

Joe M. Lara,[1] A. M. ASCE and Kenneth B. Schroder,[2] A. M. ASCE

SYNOPSIS

Two methods derived by the authors in their work with the Bureau of Reclamation are presented for the computation of water surface profiles used for the development of tail water rating curves and backwater profiles. Examples of each method are included which show the computational procedure.

INTRODUCTION

The derivation of water surface profiles serves two major purposes in the Bureau of Reclamation; one—to determine tail water rating curves, and two—to trace the backwater curve above a dam. Tail water curves are used in the design of power-plants, pumping plants, and energy dissipators such as stilling basins. In the design of large dams, these curves also furnish useful information for making stability and stress analyses. A primary use of the backwater curve above a dam is for reservoir land acquisitions and easements. Backwater information is also necessary in the design of new facilities such as pumping plants, bridges, powerplants, or other structures located along or above the reservoir.

The methods for computing water surface profiles developed herein are referred to as Method A and Method B. Method A can be used in computing profiles for channels in which the flow distance between sections is equal for all segments of the flow. Method B differs from Method A in that it is especially adaptable to conditions where the flow travel paths are longer for the main channel than for the overbank portion. Both methods, however, are based on a proper hydraulic analyses of the conveyance capacities of each cross-sectional subdivision. Eddy loss corrections and velocity head changes can also be taken into account by either method.

Theory

The underlying theory of each method is based on Bernoulli's Energy Equation which is repeated for convenience in the following discussion of its application to water surface profile computations.

The conditions of an open channel reach in steady, non-uniform flow are diagrammatically illustrated in Fig. 1. The energy balance equation is written:

$$Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_f + \text{other losses}$$

where

Z_2 = streambed elevation referenced to a given datum (upstream section)

d_2 = depth of flow at upstream section

h_{v_2} = velocity head of upstream section

Z_1 = streambed elevation referenced to a given datum (downstream section)

d_1 = depth of flow at downstream section

h_{v_1} = velocity head of downstream section

h_f = friction head loss
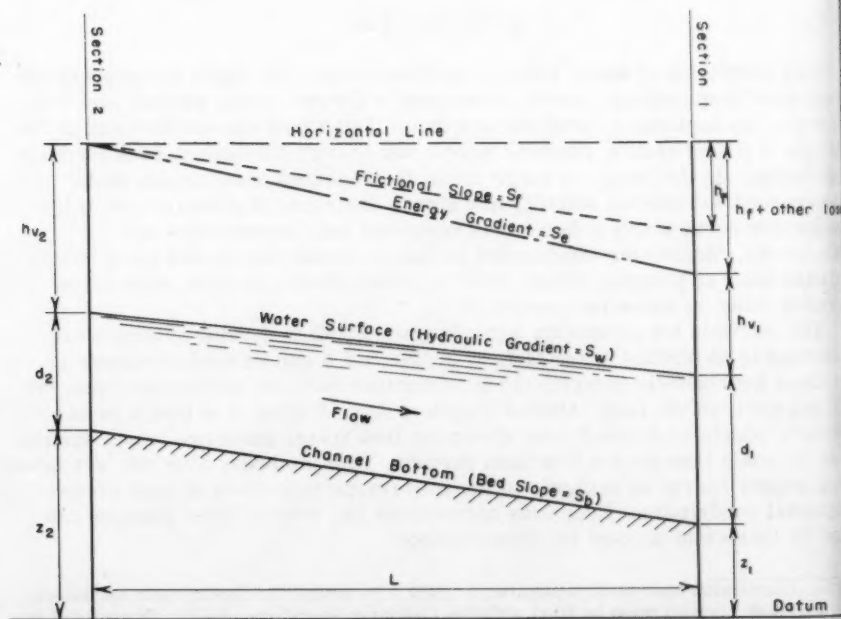
ENERGY OF OPEN-CHANNEL FLOW

FIGURE 1

Other losses = eddy, bend, and bridge pier losses.

The two methods use basically the following hydraulic relationships to evaluate the various components of the energy equation.

$$Q = AV, \quad V = Q/A, \quad h_v = \frac{V^2}{2g}$$

and

$$h_f = L \left(\frac{Qn}{1.486 \, AR^{2/3}} \right)^2$$

where

Q = discharge in cubic feet per second

A = cross-sectional area in square feet

V = cross-sectional velocity in feet per second

g = gravitational acceleration in feet per second squared

L = reach length between sections in feet

n = roughness coefficient representing the reach

R = hydraulic radius equal to cross-sectional area divided by the wetted perimeter

It is noted from above that Manning's equation is used to compute h_f. Generally, Manning's discharge equation is written as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

where S is the slope of the energy gradient.

Losses

The total losses in the energy balance equation consist of the friction head, h_f, eddy loss, bend loss, and bridge pier loss.

The friction head can be determined by applying Manning's equation in terms of the friction slope, S_f:

$$S_f = \left(\frac{Qn}{1.486 \, AR^{2/3}} \right)^2$$

The total friction head for a reach then can be resolved by the following equation:

$$h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$$

where S_{f_1} and S_{f_2} are the friction slopes computed by the preceding equation using the representative hydraulic properties: n, A, and R for Sections 1 and 2.

A simplified and practical concept of eddy loss can best be understood by studying the energy losses incident to the conversion of kinetic head to static head and vice versa in open channel flow. While there are, at present, no methods for measuring these losses, it is known they do occur and manifest

themselves usually in the form of turbulence. Observations of channel flow where there are abrupt changes in velocity clearly display turbulence by the presence of boils and vortexes. This is particularly evident when there is significant change in cross-sectional area.

It has been a general practice in the Bureau of Reclamation to compute the eddy loss as a 10-per cent correction of the difference in velocity heads (Δh_v) when the static head is converted to kinetic head and 50 per cent for a reverse condition. Thus, when the value of $h_{v_1} - h_{v_2}$ is positive, a 10-per cent correction is applied, and for negative values a 50-per cent correction is used.

Departures from the 50-per cent value quoted above may be necessary, particularly, in cases where extreme changes in velocity occur in the channel reach. This condition results in transitions from a very narrow section to a wide section. Large standing waves are formed, which indicates the flow is bordering critical stage. As the flow progresses downstream, the energy is dissipated in turbulence and upon reaching the downstream section, it reverts to a more tranquil state. In some of these cases, the eddy loss could reasonably be increased to as much as 100 per cent of the change in velocity heads. By always applying a 50-per cent correction in the computations, a situation can result showing a higher water surface elevation at the downstream section ($Z_1 + d_1 > Z_2 + d_2$, an adverse water surface slope) which is seldom encountered in natural open channel flow. To correct this condition, an adjustment can be made by increasing the percentage applied. Eddy losses, at best, are only estimates based on the judgment of the hydraulic engineer.

A search of the literature reveals that there have been no formulas developed yet for a thorough determination of bend losses. A number of formulas resulting from various studies are listed on following page.

A. $h_b = 0.21 \dfrac{\text{channel width}}{\text{inner radius}} \; h_v$ from Yarnell and Woodward[1]

B. "n" increased 0.001 for each $20°$ of curvature suggested by Scobey[2]

C. $h_b = 0.38 \, h_v$ by C. H. Yen[3]

D. $h_b = \dfrac{0.0256 \, v^2 \times \text{width of stream}}{\text{graphic radius of curvature}}$ by Boer and Urick[4]

There are no set criteria which have been established to make a practical evaluation of bend losses. Two methods are suggested for the determination of these losses; first, they could be computed from one of the formulas listed above; and second, "n" values could be modified to include bend loss effects. Ordinarily, such losses determined by the first method are very theoretical and require analyses usually beyond the limits of practical dimensions of the collected field data. It is recommended, therefore, that "n" coefficients be assigned values to include any significant effects of bend losses.

The hydraulic conditions at bridge crossings require careful analysis to determine bridge pier losses. King's Handbook[5] recommends contracted and enlargement coefficients of 0.5 and 1.0, respectively, applied to the changes in velocity heads at the approach, bridge, and emergent section. Nagler,[6] Yarnell,[7,8] and Johnson[9] have developed formulas for computing the head loss through bridges which may be applied under certain conditions.

Methods

Method A

This is a trial and error procedure which involves step-by-step computations. The method has a very wide scope of application and can be used in the general case involving steady, non-uniform flow. It is particularly applicable to the irregular channel in which the cross section consists of a main channel and separate overbank areas having individual "n" coefficients. Velocity head changes are taken into account by a weighting process and corrections can be included for eddy losses within the reach. Reach lengths representing the flow path between sections, however, are assumed equal for the main channel and overbank areas.

Referring to the diagrammatic sketch in Fig. 1, the objective is to determine the water surface elevation for a given discharge at Section 2 when it is known at Section 1.

When the discharge exceeds main channel capacity, the flow also inundates the flood plain or overbank area. If channel bed conditions are different from those of the overbank areas, separate "n" values are assigned, and the cross section is subdivided for proper hydraulic analysis.

The areas and hydraulic radii are computed separately for each subdivision and tabulated as shown for Section 1 in Table 1. The conveyance capacity, K_d, equal to $1.486/n \ AR^{2/3}$, also shown in this table, is computed for the main channel and overbank using the corresponding "n" value. Conveyance curves, K_d vs. elevation, are plotted for each section; an example is shown in Fig. 2.

In this method, the friction slope, S_f, is determined by the following analysis:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \qquad \text{(Manning)}$$

since

$$K_d = \frac{1.486}{n} AR^{2/3}$$

then

$$Q = K_d S_f^{1/2}$$

solving for S_f

$$S_f = \left(\frac{Q}{K_d}\right)^2$$

The K_d value used in the above equation is the total conveyance capacities of the main channel and overbank areas.

The discharges occurring in each subdivision of the cross section are determined as follows:

$$Q = K_d S_f^{1/2}$$

and

$$Q_p = K_{d_p} S_f^{1/2}$$

where

Q = total discharge in cfs

HYDRAULIC CHARACTERISTICS

Section No. 1   Main Channel

n = 0.030   1.486/n = 49.63

Elevation	A	R	$R^{2/3}$	Sub K_d	Total K_d
5701	14.5	0.48	0.613	440	440
5702	52	1.27	1.173	3,020	3,020
5707	263	4.99	2.920	38,000	38,000
5711	449	7.21	3.732	83,000	83,000
5713	547	8.17	4.057	110,000	110,000
5713.5	570	8.55	4.181	118,000	118,678
5714	594	8.77	4.253	125,000	129,150

Overbank Area A_1

n = 0.050   1.486/n = 29.72

| 5713.5 | 57.5 | 0.25 | 0.397 | 678 |
| 5714.0 | 192 | 0.62 | 0.727 | 4,150 |

TABLE 1
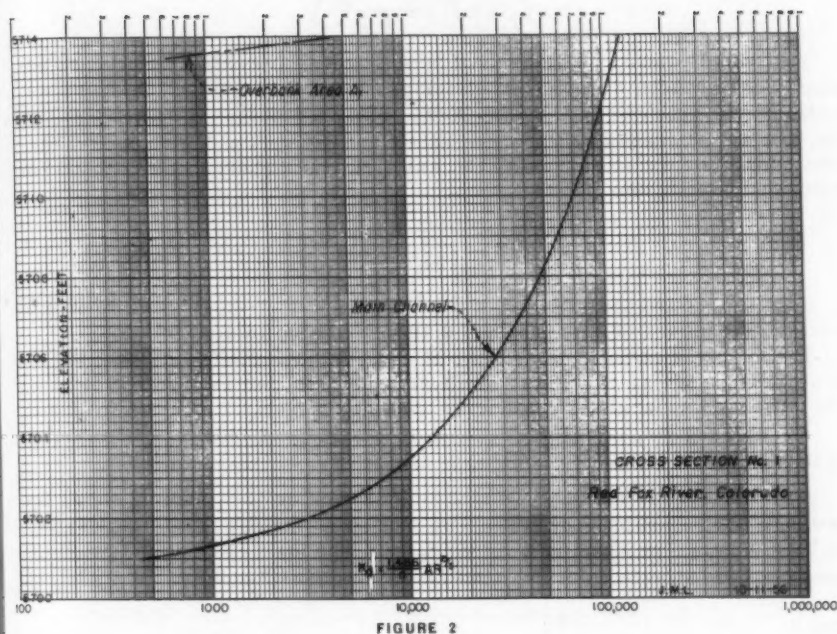
FIGURE 2

S_f = friction slope

K_d = total conveyance capacity

and Q_p and K_{d_p} = corresponding elements for partial discharge and conveyance capacity of the subdivision under consideration.

Assuming a unit slope in the above equations, they are written thus:

$$Q = K_d \times 1 \quad \text{and} \quad Q_p = K_{d_p} \times 1$$

dividing one equation by the other

$$\frac{Q}{Q_p} = \frac{K_d}{K_{d_p}}$$

solving for Q_p

$$Q_p = \frac{Q \times K_{d_p}}{K_d}$$

From the preceding analysis, it is noted that the K_d value, in addition to its definition as conveyance capacity, can also be designated as the discharge occurring with a slope equal to unity.

The friction head, h_f, is determined by averaging the slopes at Sections 1 and 2 then multiplying by the length as illustrated below:

$$h_f = L \left(\frac{S_{f_1} + S_{f_2}}{2} \right)$$

The velocity head, h_v, is derived by a weighting process using the partial discharges occurring in each subdivision of the cross section. This method is identical to the one used by the Corps of Engineers.[10] Velocities in each segment are computed by the basic equation, $V_p = Q_p/A_p$, where A_p is the area of the particular segment. The weighted velocity head results from the following relationship:

$$h_v = \left[\frac{\Sigma(V_p^2 \, Q_p)}{2gQ} \right]$$

Eddy losses are computed from the formula $M(h_{v_1} - h_{v_2})$ where M is 0.50 for negative values of $h_{v_1} - h_{v_2}$ and 0.10 for positive values.

The following example illustrates an application of Method A using the data of a fictitious stream, the Red Fox River, Colorado, for a discharge of 11,100 cfs. The column numbers refer to the columns in Table 2.

Step 1

Enter the cross-section designation under Column 1. Column 2 shows the assumed water surface elevation. For Section 1, this has been established as 5714.0 from a predeveloped rating curve. The main channel and overbank areas and conveyance values are entered in Columns 3 and 5.

Step 2

The friction slope, S_{f_1}, is computed from the equation $S_f = (Q/K_d)^2$ as shown in Column 6. Thus, in the example, this is equal to $(11,100/129,150)^2 = 0.00755$. Note that the total K_d (Column 5) is used.

Step 3

Column 9 shows the subdivisional discharges as computed by the equation

$$Q_p = \frac{Q \, K_{d_p}}{\Sigma K_d}$$

Substituting the values of the main channel in the example:

$$Q_p = \frac{(11,100)(125,000)}{129,150} = 10,700$$

A similar computation is made for the overbank area using its K_d value of 4,150.

Step 4

The velocities in Column 10 are obtained by dividing Column 9 by Column 5 for each subdivision. Velocity in the main channel equals 10,700 divided by 594.

Step 5

Column 11 is computed by squaring Column 10 and multiplying by Column 9. The values of each subdivision are totaled and entered.

WATER SURFACE PROFILE COMPUTATIONS - METHOD B

SUBJECT: Silver Fox River   Discharge = 50,000 cfs
COMPUTED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____ SHEET 1 OF 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SECTION	Reach Length L	Assumed W S Elevation	Area A	$K_d = \frac{1.486\,AR^{2/3}}{n}$	$K_d / L^{1/2}$	$\left(\frac{Q}{\Sigma \frac{K_d}{L^{1/2}}}\right)^2 = h_f$	$\left(\frac{K_d}{L^{1/2}}\right)(h_f)^{1/2} = Q$	V	$V^2 Q$	h_v	$h_{v_1} - h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	ΔH	Water Surface Elevation
209+40	2,260	94.67	3,390	973,000	20,500		30,000	8.85	2,350,000							94.67
	1,255		11,600	664,000	13,700		20,000	1.72	59,000							
					34,200	2.13	50,000		2,409,000	0.75						
186+80	2,260	96.90	4,140	1,070,000	22,600		28,700	6.93	1,380,000							96.88
	1,955		13,650	739,000	16,700		21,300	1.56	52,000							
					39,300	1.68	50,000		1,432,000	0.45	+0.30	0.03	1.88	1.91	2.21	96.88
186+80	3,870	96.90	4,140	1,070,000	17,800		28,600	6.91	1,365,000							
	3,265		13,650	739,000	12,900		21,400	1.57	53,000							
					30,100	2.76	50,000		1,418,000	0.44						
148+10	3,870	100.35	4,660	1,258,000	20,300		45,700	2.80	4,390,000							100.35
	3,265		3,520	110,200	1,920		4,300	1.22	6,400							
					22,120	5.10	50,000		4,396,400	1.37	-0.93	0.47	3.31	4.40	3.47	100.35
148+10	2,300	100.35	4,660	1,258,000	26,200		45,500	9.76	4,330,000							
	1,800		3,520	110,200	2,600		4,500	1.28	7,360							
					28,800	3.01	50,000		4,337,360	1.35						
125+10	2,300	101.40	4,520	1,360,000	28,100		21,000	4.64	453,000							101.45
	1,800		20,700	1,642,000	38,800		29,000	1.40	26,900							
					66,900	0.56	50,000		509,200	0.16	+1.19	0.12	1.79	1.91	3.10	101.45
125+10	2,880	101.40	4,520	1,360,000	25,100		21,100	4.67	460,000							
	2,290		20,700	1,642,000	34,400		28,300	1.40	56,400							
					59,500	0.71	50,000		516,400	0.16						
96+30	2,880	106.50	4,440	1,238,000	23,100		29,800	6.71	1,340,000							104.51
	2,290		13,100	759,000	15,700		20,200	1.54	48,000							
					38,800	1.66	50,000		1,398,000	0.43	-0.27	0.14	1.19	1.33	1.06	104.51
96+30	5,755	106.50	4,440	1,238,000	16,300		30,200	6.80	1,400,000							
	4,980		13,100	759,000	10,700		19,800	1.51	45,000							
					27,000	3.42	50,000		1,445,000	0.45						
38+75	5,755	111.10	3,840	918,000	12,100		39,300	10.22	4,020,000							111.10
	4,980		5,900	232,000	3,310		10,700	1.81	35,000							
					15,410	10.55	50,000		4,055,000	1.26	-0.81	0.41	6.99	7.40	6.59	111.10

TABLE 3A

Step 6

The weighted velocity head, h_v, is computed from the following relationship:

$$h_v = \frac{\Sigma (v^2 \, q)}{2gQ}$$

Substituting in the example:

$$\frac{3,461,730}{64.4 \times 11,100} = 4.85$$

which is entered in Column 12.

Step 7

The distance between sections is entered in Column 4.

Step 8

An elevation is assumed and entered in Column 2 for Section 2. Similar entries are made in Columns 3 and 5 as was done for Section 1 described in Step 1 above.

Step 9

The friction slope is computed as described under Step 2 and entered in Column 6. Column 7 shows the average of the consecutive slopes as follows:

$$\frac{0.00735 + 0.000673}{2} = 0.00401$$

Column 8, the friction head, equals Column 7 multiplied by Column 4.

Step 10

Columns 9, 10, 11, and 12 are computed as described in Steps 3, 4, 5, and 6.

Step 11

The algebraic difference in velocity heads is entered in Column 13. In the example, this equals $4.85 - 0.43 = 4.42$.

Step 12

The eddy loss Column 14 is taken as 10 per cent of Column 13 for positive values or 50 per cent for negative values. This equals $(0.10)(4.42) = 0.44$ in the example.

Step 13

Column 15 is the summation of Columns 8 and 14. Column 16 equals Columns 15 plus 13.

Step 14

Column 17 is obtained by adding the water surface of the downstream section (Column 17—Section 1) to the value shown in Column 16. For example, this equals $5714.0 + 6.86 = 5720.86$. When the result in Column 17 is equal to

Column 2 to the nearest tenth foot, the computations are begun at the next upstream section.

Method B

This method is also a trial and error procedure involving step computations. It is used in cases involving steady, nonuniform flow. However, it differs from Method A in that the reach lengths representing the flow path between sections are different for the main channel and overbank. The overbank reach length could be considerably shorter. Consideration is also given to the variability of the hydraulic elements "n", area, and hydraulic radius as was done in Method A.

Again, the sketch in Fig. 1 representing the energy equation is basic to this method. Knowing the elevation at Section 1, the water surface elevation at Section 2 is assumed and the energy equation checked.

The determination of the friction head, h_f, will be discussed first. This is accomplished by applying Manning's formula as follows:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

expressing S_f as $\frac{h_f}{L}$ where L is the length between sections. Substituting

$$Q = \frac{1.486}{n} AR^{2/3} \left(\frac{h_f}{L} \right)^{1/2}$$

Substituting K_d in the equation

$$Q = K_d \left(\frac{h_f}{L} \right)^{1/2}$$

squaring and solving for h_f

$$h_f = \left[\frac{Q}{\frac{K_d}{L^{1/2}}} \right]^2$$

however, since there are two or more values for $K_d/L^{1/2}$ computed because of the differences in flow paths and conveyance capacities for the main channel and overbank areas, the summation (Σ) of $K_d/L^{1/2}$ is used:

$$h_f = \left[\frac{Q}{\Sigma \left(\frac{K_d}{L^{1/2}} \right)} \right]^2$$

A mean friction head results from the equation $h_f = \frac{h_{f_1} + h_{f_2}}{2}$ where h_{f_1} and h_{f_2} are the friction heads computed for Sections 1 and 2, respectively.

The discharges occurring in each sectional subdivision are computed from the formula listed below:

$$Q_p = \left(K_d/L^{1/2} \right) h_f^{1/2}$$

Velocity heads and eddy losses are treated the same as in Method A.

An application of Method B is shown below using the data of a fictitious stream, Silver Fox River for a discharge of 50,000 cfs. Column numbers refer to the columns in Table 3.

WATER SURFACE PROFILE COMPUTATIONS – METHOD B

SUBJECT: Silver Fox River — Discharge = 50,000 cfs

COMPUTED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____ SHEET 2 OF 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
SECTION	Reach Length L	Assumed W S Elevation	Area A	$K_d = \frac{1.486}{n} AR^{2/3}$	$K_d/L^{1/2}$	$\left(\frac{Q}{\Sigma}\frac{K_d}{L^{1/2}}\right)^2 = h_f$	$(K_d/L^{1/2})(h_f)^{1/2} = Q$	V	V^2Q	h_v	$h_{v_1}\!-\!h_{v_2}$	Eddy Loss	Mean h_f	Total Loss	ΔH	Water Surface Elevation
38-75	3,875	111.10	3,840	918,000	14,750		39,200	10.20	4,080,000							
	3,825		5,990	232,000	4,040	7.08	10,800	1.81	36,000	1.28						
					18,790		50,000		4,116,000							
0-00	3,875	116.60	4,130	1,128,000	19,200		28,800	6.98	1,405,000							
	3,295		14,020	807,000	14,100	2.26	21,200	1.51	48,000	0.45	+0.83	0.08	4.67	4.75	5.58	116.63
					33,300		50,000		1,453,000							

TABLE 3 B

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Red Fox River, Colorado — Discharge = 11,100 cfs

COMPUTED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____ SHEET ___ OF ___

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	Assumed	Area	Dist		1.486			Q =		Mean Su...						

TABLE 3B

WATER SURFACE PROFILE COMPUTATIONS – METHOD A

SUBJECT: Red Fox River, Colorado   Discharge = 11,100 cfs

COMPUTED BY: ___ DATE ___ CHECKED BY: ___ DATE: ___ SHEET ___ OF ___

1 STATION	2 Assumed Water Surface Elev.	3 Area A	4 Dist. L	5 $K_d = \frac{1.486 AR^{2/3}}{n}$	6 $S_f\left(\frac{Q}{K_d}\right)^2$	7 Mean S_f	8 Mean $S_f \times L = h_f$	9 Q	10 V	11 V^2Q	12 h_v	13 $h_{v_1} - h_{v_2}$	14 Eddy Loss	15 Total Loss	16 ΔH	17 W.S. El.
Section 1	5714.0	594		125,000				10,700	18.0	3,460,000						5714.0
		192		4,150				400	2.08	1,729						
				129,150	0.00735			11,100		3,461,730	4.85					
Section 2	5720.9	863	500	230,000				5,960	6.80	276,000						5720.86
		1,170		130,000				3,370	2.88	28,000						
		1,000		25,000				650	0.65	274						
		1,500		43,000				1,120	0.75	630						
				428,000	0.000673	0.00401	2.00	11,100		304,904	0.43	+4.42	0.44	2.44	6.86	
Section 3	5721.2	1,832	400	360,000				11,000	6.00	396,000						5721.15
		115		2,700				70	0.61	26						
		185		1,250				30	0.16	8						
				363,950	0.00108	0.000877	0.35	11,100		396,034	0.56	-0.13	0.07	0.42	0.29	
Section 4	5721.2	1,300	100	158,000	0.00493	0.00301	0.30	11,100	8.54		1.13	-0.57	0.27	0.57	0.00	5721.20
Section 5	5722.1	1,681	100	220,000	0.00254	0.00374	0.37	11,100	6.60		0.68	+0.45	0.05	0.42	0.87	5722.07
Section 6	5722.8	1,018	400	174,000				10,900	10.7	1,250,000						5722.85
		130		3,380				190	1.46	405						
		20		161				10	0.50	3						
				177,541	0.00391	0.00328	1.31	11,100		1,250,408	1.75	-1.07	0.54	1.85	0.78	
Section 7	5725.3	844	500	124,000				10,500	12.5	1,640,000						5725.35
		98		1,400				110	1.12	138						
		220		5,800				490	2.23	2,440						
				131,200	0.00714	0.00551	2.77	11,100		1,642,578	2.30	-0.55	0.28	3.05	2.50	

TABLE 2

Step 1

Enter the pertinent data in the heading of Table 3. In the example, these are the Silver Fox River water surface profile computations for a discharge of 50,000 cfs.

Step 2

Enter the station number 209+40 under Column 1.

Step 3

Under Column 2, the main channel and overbank reach lengths between sections are entered. These are noted as 2,260 and 1,955 feet between Sections 209+40 and 186+80 for the main channel and overbank reaches.

Step 4

The known water surface elevation of 94.67 is entered in Column 3 for Section 209+40. Areas of each subdivision are entered in Column 4.

Step 5

K_d values of 973,000 and 604,000 are entered in Column 5 for the main channel and overbank area, respectively.

Step 6

Column 6 is determined by dividing the individual values of Column 5 by the square root of Column 2. For example, the main channel value is computed as follows:

$$\frac{K_d}{L^{1/2}} = \frac{973,000}{(2,260)^{1/2}} = 20,500$$

Step 7

The friction head, h_f, in Column 7 is found by squaring the quantity of the total discharge divided by the summation of the values in Column 6, as follows:

$$h_f = \left(Q/\Sigma \frac{K_d}{L^{1/2}} \right)^2 = \left(\frac{50,000}{34,200} \right)^2 = 2.13$$

Step 8

The subdivisional flows listed in Column 8 are computed by multiplying the individual values of Column 6 by the square root of Column 7. For example, the overbank discharge is computed as follows: $13,700 \times 2.13^{1/2} = 20,000$.

Step 9

Columns 9, 10, and 11 are determined as shown in Steps 4, 5, and 6 under Method A.

Step 10

The ... ginning ... surface ... sumed a ... Column ... 186+80 ... entered

Step 11

Eddy

Step 12

Colu ... values i

Step 13

Colu ... algebra

Step 14

The ... 16 to th ... the exa ... nearest ... are con

The ... Section ... and 3,2 ... elevatio ... is begu

---

1. Woo ... Bull

2. Scol ... 393,

3. Yen ... Can

4. Boe ... bend ... 109,

5. King ... Fou

Step 10

The remainder of the columns are omitted since Section 209+40 is the beginning section. The next step, therefore, consists of assuming the water surface elevation for the next upstream Section 186+80. This has been assumed as 96.90 in the example. Similar computations are made through Column 11 as described for Section 209+40. The velocity head, h_f, for Section 186+80 computes to 0.45. The algebraic difference in velocity heads is entered in Column 12 as in the example $h_{v_1} - h_{v_2} = 0.75 - 0.45 = +0.30$.

Step 11

Eddy losses in Column 13 are determined as in Step 12 of Method A.

Step 12

Column 14 is the mean friction head loss, h_f, computed by averaging the values in Column 9 as shown in the following computations:

$$h_f = \frac{2.13 + 1.62}{2} = 1.88$$

Step 13

Column 15 is equal to the total of Columns 13 and 14. Column 16 is the algebraic sum of Columns 12 and 15.

Step 14

The water surface elevation in Column 17 is calculated by adding Column 16 to the preceding water surface elevation (Section 209+40) of Column 17. In the example, this amounts to $94.67 + 2.21 = 96.88$. When a balance to the nearest tenth foot is obtained between Columns 17 and 3, the computations are complete.

The next step involves a computation of the friction head, h_f, between Sections 186+80 and 148+10 (next upstream section) using the lengths of 3,870 and 3,265 as shown for the main channel and overbank. The water surface elevation at Section 148+10 is then assumed and a new cycle of computations is begun using the same process outlined above.

BIBLIOGRAPHY

1. Woodward, Sherman M., "Flow of Water around 180° Bends," Technical Bulletin No. 526, October 1936, U. S. Department of Agriculture.

2. Scobey, Fred C., "The Flow of Water in Flumes," Technical Bulletin No. 393, U. S. Department of Agriculture.

3. Yen, C. H., and J. W. Howe, "Effects of Channel Shape on Losses in a Canal Bed," Civil Engineering, January 1942.

4. Boer, B. C., and R. H. Urick, (title unknown). An experimental study of a bend of the Iowa River made in 1913-1914. Quoted from page 627, Volume 109, Transactions, American Society of Civil Engineers, 1944.

5. King, Horace W., revised by Ernest F. Brater, "Handbook of Hydraulics," Fourth Edition, McGraw-Hill Book Company, 1954.

6. Nagler, F. A., "Obstruction of Bridge Piers to the Flow of Water,"
    Volume 82, Transactions, American Society of Civil Engineers, 1918.

7. Yarnell, David L., "Bridge Piers as Channel Obstructions," Technical
    Bulletin No. 442, U. S. Department of Agriculture.

8. Yarnell, David L., "Pile Trestles as Channel Obstructions," Technical
    Bulletin No. 429, U. S. Department of Agriculture.

9. Johnson, Hollister, "Computation of Peak Discharge by the Contracted-
    Opening Method," unpublished report, U. S. Department of the Interior,
    Geological Survey. A similar report has been published by the U. S.
    Department of the Interior, "Computation of Peak Discharge at Con-
    tractions," Geological Survey Circular No. 284, 1953, C. E. Kindsvator,
    R. W. Carter, and H. J. Tracy.

10. "Hydrologic and Hydraulic Analyses—Computation of Backwater Curves
    in River Channels," Engineering Manual, Department of the Army, Corps
    of Engineers, Part CXIV, Chapter 9, May 1952.

Journal of the
HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

Page

TRANSITION FROM LAMINAR TO TURBULENT FLOW IN A PIPE[a]

Closure by M. R. Carstens

M. R. CARSTENS,[1] A. M. ASCE.—The discussion[1] by Prof. Robertson is a thorough review of the paper in which valid questions were posed concerning the interpretation of the experimental results.

The most important question of the discussion concerned the propriety of using the Blasius boundary layer in the discussion of stability of the temporally developing boundary layer in a pipe. Specifically, the question was raised as to whether or not boundary-layer stability studies by Tatsumi[2] would be more applicable. Neither the Blasius boundary layer nor the Tatsumi boundary layer is the time developing boundary layer in pipe, since the Blasius boundary layer applies to steady-flow over a flat plate without a pressure gradient and the Tatsumi boundary layer applies to steady flow in a pipe in the inlet region. In spite of the differences of origin, the Tatsumi boundary layer and the Blasius boundary layer are remarkably similar, as shown in Fig. I in which y is the distance from the boundary. Since the second derivative of v with respect to y is important in stability analyses, velocity profiles may appear to be quite similar as in Fig. I but result in radically different neutral stability curves. Thus, further evidence of the remarkable similarity between the Tatsumi and Blasius boundary layers is shown in Fig. II in which a neutral stability curve for the Tatsumi boundary layer is superposed upon the neutral stability curve of the Blasius boundary layer. Therefore, there appears to be no intrinsic merit in using the Tatsumi boundary layer for qualitative discussion of a case for which the stability analysis is not available. On the other hand, the advantage of using the Blasius boundary layer for comparison is that the stability analyses are much more comprehensive than for any other boundary-layer velocity profile.

The comparison of the stability of the temporally developing boundary layer in a pipe can also be made with the stability of the Polhausen boundary layer.[3] In Fig. III the plotted points were obtained from an analog computer solution of the Navier-Stokes equation with boundary conditions and physical constants of Run 29. The analog computer solution for the velocity profiles shown in Fig. III is in the range of boundary-layer thickness for which transition to turbulence was observed. The ratio of the displacement thickness, δ^*, to the momentum thickness, Θ, is about 2.43 for the four velocity profiles shown in Fig. III. This value of δ^*/Θ corresponds to a Polhausen shape parameter of 3. Stability analysis of this Polhausen boundary layer indicates that the critical or minimum value of the boundary-layer Reynolds number,

a. Proc. Paper 1450, December, 1957, by M. R. Carstens.
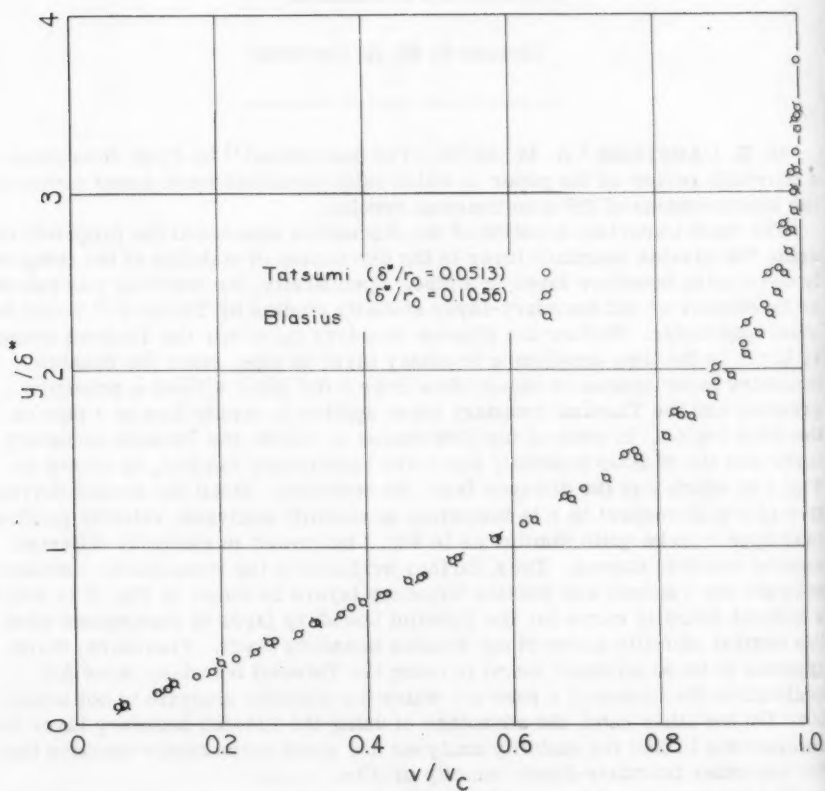1. Prof. of Civ. Eng., Georgia Inst. of Technology, Atlanta, Ga.

Figure I.   Comparison of the Blasius and Tatsumi Boundary-
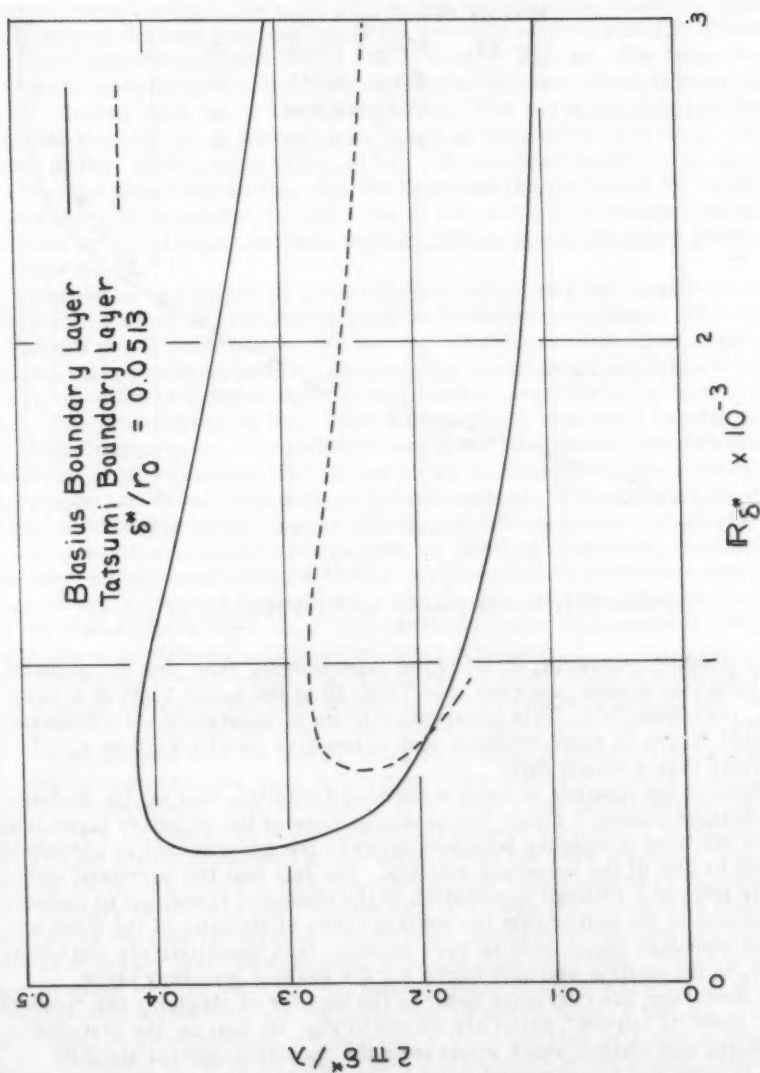Layer Velocity Profiles

Figure II. Comparison of Neutral Stability Curves of the
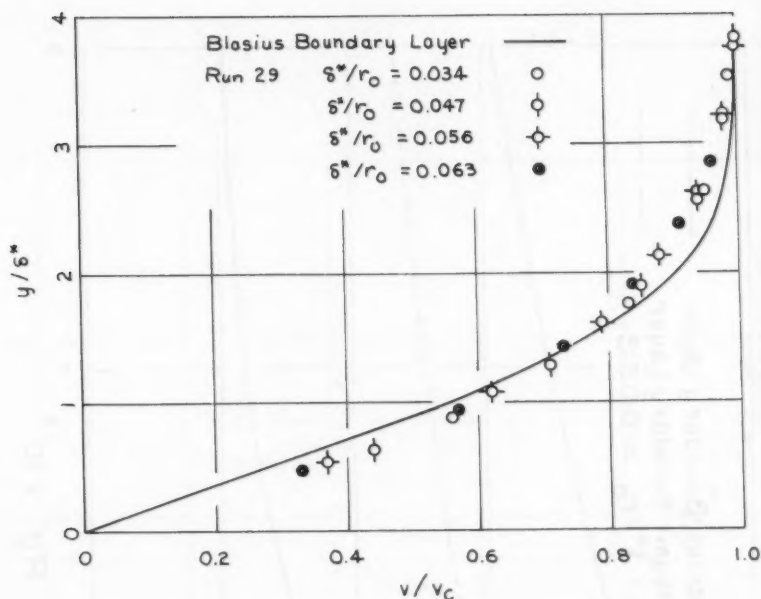   Blasius and Tatsumi Boundary Layers

Figure III. Boundary-Layer Velocity Profiles of Run 29

$\mathbb{R}_{\delta*}$ is 3500.[4] However, in all of the experimental runs, the inception of turbulence was always observed (see Table III of the paper 1450) at a value of $\mathbb{R}_{\delta*}$ less than 3500. This observation tends to substantiate the discusser's statement "there is some evidence that a timewise developing flow may be less stable than a steady flow."

In spite of the hazards of using a borrowed solution, that is, the stability of the Blasius boundary layer, the general nature of the boundary layer instability in the time developing boundary layer of the experiments is undoubtedly identical to that of the borrowed solution. The fact that the borrowed stability analysis offered a rational explanation of the observed transition to turbulence is indicative to the author that the neutral curve of stability of the time developing boundary layer must be very similar, both quantitatively and qualitatively, to the neutral stability curve for the Blasius boundary layer.

The discusser has correctly deduced the manner of obtaining the "possible neutral stability curves" which are shown in Fig. 10, that is, the disturbance wave length was chosen which would pass the "possible neutral stability curve" through the experimental point having the highest transition Reynolds number. The hypothesis that the disturbance wave length, λ, is proportional to the roughness height was based upon an analogy with the Karman vortex trail behind a circular cylinder. In a stable Karman vortex trail, the spacing between successive vortices is proportional to cylinder diameter and independent of the flow variables. The analogy is suggestive that the disturbance wave length, λ, is proportional only to the roughness height or, in this case, the step height of the mismatched pipe boundaries at the junction. The mismatching of the pipe ends could not be expected to be axisymmetric. Thus,

many disturbance wave lengths, ranging downward in magnitude from the maximum wave length at the point of greatest mismatching, could be expected to originate at the pipe junction. Only the possible neutral stability curve for the longest disturbance wave length was plotted on Fig. 10. The other shorter disturbance wave lengths originating at the pipe junction would have neutral stability curves lying below the plotted curve. The curve defining the limits of neutral stability of all disturbances would be bounded by a vertical line on the left in Fig. 10 at a value of $\mathbb{R}_{\delta *}$ of 420 with an upper boundary as shown in Fig. 10. The above explanation was the basis for the statement "With this concept there is no conflict in that some of the points of turbulence inception (indicated by the circular symbols on Fig. 10) are below the lower branch of the curve shown."

The question was raised in the discussion concerning the amplification of a disturbance which is required to produce turbulence inception. The stability analyses of small disturbances are merely indications as to whether the disturbance will amplify or decay. The stability analyses do not indicate whether the amplification of a disturbance is sufficient to result in transition to turbulence. The presumption is that, when a disturbance reaches a certain magnitude, transition occurs. The order of magnitude of amplification required for transition is not a constant, as implied by the discusser, but must certainly depend upon the initial magnitude of the disturbance. For example, an infinitesimal disturbance would require infinite amplification and a finite disturbance, which is equal or greater than the limiting magnitude, would cause transition without amplification. Since the transition to turbulence was always observed some distance downstream from the pipe junction, the conclusion is that the disturbances were small (but certainly finite) disturbances that required some amplification before transition occurred.

The discussion concerning Fig. 7 appears to result from a misunderstanding. A brief review of the total transition occurrence will clarify the meaning of this figure. Prior to opening of the downstream valve, the fluid is at rest throughout the pipe. At the instant of valve opening a piezometric-pressure gradient is applied to the fluid in the pipe. The unbalanced force system results in acceleration. The developing flow pattern can be visualized as a central core with uniform velocity distribution and a boundary layer in which the velocity varies from zero at the boundary to the core velocity. The core velocity increases linearly with time during the initial stages of the motion. The boundary layer is of zero thickness at $t = 0$ and increases with time. The flow is laminar throughout. Meanwhile, small disturbances are being fed into the boundary layer downstream from the pipe junctions. There is only a limited range of core velocity, v_c, and boundary-layer thickness, δ^*, during which these disturbances will be amplified as shown Figs. 9 and 10. During this period of amplification, a disturbance from the pipe junction is amplified until a turbulent burst is formed. Once a turbulence burst exists, large adjustments in the laminar velocity profile are effected. Since the pipe flow is limited in extent, these adjustments within the boundary layer also result in appreciable velocity changes in the central core adjacent to the turbulent burst. Since the concept of a laminar central core riding over a turbulent boundary layer appears to be untenable, the turbulence must penetrate into the central core in some manner. Whether the velocity adjustment leads to instability in the central core or whether the disturbance of the turbulent burst is sufficient to cause instability and a radial spread of turbulence is not known to the author. In any event the schematic drawing in Fig. 7, which is

based upon a study of the film strip data and pressure data as the downstream face of the turbulent spot passed through the pipe outlet, appears to be a rational picture of the downstream face of a turbulent spot which moves toward the pipe outlet. Essentially, the same type of downstream face of a turbulent spot is reported by Schubauer and Klebanoff[5] for flow over a flat plate as follows:

> "The turbulence is, in fact, transported downstream with the free-stream velocity and the lag at the surface is due to the time required for propagation inward through the slower moving air of the laminar layer. Probably the chief significance of the slower progress near the surface is that it gives rise to an overhanging leading edge."

Two errors in the paper were pointed out in the discussion. First, the number of wave lengths between the point of disturbance origin and the point of turbulence inception would be about 4 to 37 for turbulent spot 2 and about 13 to 31 for turbulent spot 3 based on wave lengths of 2.1 r_0 and 1.5 r_0, respectively. Second, the X/D value for spot 1 of Run 20 should be 12 rather than 112 in Table III. A question mark in parenthesis should be placed in Table III adjacent to X/D value of spot 1 of Run 21.

The author thanks Professor Robertson for his carefully prepared discussion of the paper and thanks Messrs. Johnson and Meeks of the Georgia Tech Analog Computer Section for their help in obtaining the solutions shown in Fig. III.

REFERENCES

1. Proc. Paper 1856, November, 1958, by J. M. Robertson.

2. Tatsumi, Tomomasa, "Stability of the Laminar Inlet-Flow prior to the Formation of Poiseuille Régime," Journal of the Physical Society of Japan, Vol. 7, No. 5, Sept.-Oct., 1952, pp. 489-502.

3. Schlichting, H., Boundary Layer Theory, Pergamon Press, 1955, pp. 206-213.

4. Ibid., p. 341.

5. Schubauer, G. B. and Klebanoff, P. S., "Contributions on the Mechanics of Boundary-Layer Transition," NACA, TN 3489, September 1955.

SOME EXPERIMENTS WITH EMERGENCY SIPHON SPILLWAYS[a]

Discussion by Fred W. Blaisdell and Harold W. Humphreys

FRED W. BLAISDELL,[1] M. ASCE and HAROLD W. HUMPHREYS,[2] A. M. ASCE.—India, in addition to the countries listed by the author, has used large siphon spillways for the close control of reservoir levels. But only India, to the writers' knowledge, has used both "saddle" and "volute" siphons,[1] as they call them. This discussion will be confined largely to the volute siphon and various modifications of it.

The volute siphon might be defined as a morning glory spillway which has the entrance covered by a dome. The dome is of greater diameter than the entrance and extends below the crest elevation in the same manner as does the top or hood of the saddle siphon. Also, like the saddle siphon, the volute siphon uses siphon breakers which run to the top of the dome. In contrast to the saddle siphon as detailed by Mr. McBirney, the volute siphon does not curve the lower leg upstream or necessarily use a device to throw the nappe to the crown, nor is it necessary to submerge the exit to insure priming.

Indian literature credits Shri V. Ganesh Iyar with the discovery of the volute siphon in 1933 and its experimental development at Mandya in 1935-36 using a siphon 2.5 feet in diameter and 11 feet high.[1,2] Two volute siphons eight feet in diameter with an operating head of 43 feet were constructed at Marconahally in 1938 as an experimental measure. Eleven siphons 18 feet in diameter with a drop from the design head-water level to the invert at the spillway exit of 68 feet were built in 1947 at Hirebhasgar.

It is not necessary to use the dome to insure that the spillway will run full under suction. Examples of this may be found in the Heart Butte, North Dakota, and Shade Hill, South Dakota, spillways of the U. S. Bureau of Reclamation.[3] If a siphon is defined as a spillway which requires that the liquid be lifted above the headpool level, then the Dakota spillways do not qualify as siphons. They do qualify as siphons if the definition is:

A closed conduit, a portion of which lies above the hydraulic grade line. This results in a pressure less than atmospheric in that portion, and hence requires that a vacuum be created to start flow. A siphon utilizes atmospheric pressure to effect or increase the flow of water through it.

---

a. Proc. Paper 1807, October, 1958, by Warren B. McBirney.
1. Project Supervisor, Agri. Research Service, U. S. Dept. of Agri., St. Anthony Falls Hydr. Lab., Minneapolis, Minn.
2. Head, Hydr. Research Section, State Water Survey, Urbana, Ill.

The Dakota spillways qualify as siphons because, under full flow, the vertical legs are under pressures less than atmospheric and lie above the hydraulic grade line.

The possibility that the pressures within these spillways may be so low as to cause cavitation should be recognized. The Hirebhasgar volute siphons[1] in India are examples. These siphons consist of a dome-covered morning glory entrance 30 feet 8 inches in diameter which converges to a drop inlet shaft 18 feet in diameter, a 90 degree elbow, and a horizontal leg 18 feet in diameter. The drop in level between the crest and the exit center line is about 57 feet—considerably in excess of absolute zero pressure head even considering hydraulic losses. Cavitation damage should have been expected and did occur.[1] Cavitation damage also has occurred in the Marconahally siphons. This should have been anticipated since one-sixteenth size model tests indicate prototype pressures below absolute zero over much of the drop inlet and inside of the elbow.[5]

In the Dakota spillways of the U. S. Bureau of Reclamation, the possibility of extremely low pressures was recognized and means were taken to prevent their occurrence. In the case of the Heart Butte spillway, a lip is used to cause the stream to break away from the wall of the drop inlet. This insures atmospheric pressure at that point and that negative pressures in the drop inlet will not exceed a predetermined magnitude. The drop through the Shade Hill spillway does not appear to be sufficient to cause cavitation problems.

Full flow can be achieved for spillways of the morning glory type without special priming devices if the inlet is properly formed. The writers are of the opinion that the Hungry Horse spillway[3] would flow full if the head over the crest is raised high enough. If this were to occur, serious cavitation would be a likelihood, in view of the 487 foot drop through the spillway, unless means were taken to prevent it. Presumably the maximum head over the crest is sufficiently low and the barrel sufficiently large to insure part full flow at all times. It is important that designers recognize that morning glory spillways can flow full, in order to know what to design against if they want part full flow, or to know what to design for if they want full flow.

The Dakota spillways required no priming or depriming aids other than shaping the inlet properly. The head-discharge curves for the Heart Butte spillway presented by Peterka[6] show that the flow through the spillway is controlled by a weir at low heads. At high heads, the control is by pipe flow when the inlet is submerged and the drop inlet is full. At the higher weir heads, the falling water must partially seal off the drop inlet and the increased head must create a demand for more water than can be supplied over the crest weir at that headpool level. The excess demand is supplied by air entrainment and air flow through the spillway. This is, the writers believe, the partialization for which Mr. McBirney is looking. Partialization is achieved here without special devices.

Only relatively large spillways have been discussed up to this point. Large closed conduit spillways designed to flow full, like the Heart Butte and Shade Hill structures, are few in number. However, the U. S. Soil Conservation Service (SCS) began building this type of spillway in relatively smaller sizes in the 1930's. Closed conduit self-priming spillways built annually under Public Law No. 46, 74th Congress, entitled "An Act to Provide for the Protection of Land Resources Against Soil Erosion, and for other Purposes;" and Public Law 566, 83rd Congress, known as the Watershed Protection and Flood

Prevention Act, number in the thousands[7] and many of them operate at design capacity each year. The hydraulic design of these SCS spillways is identical to the design methods which should be used for the relatively larger spillways of the morning glory type. The senior writer has outlined the theory previously[8] and recently has authored a series of papers[9] describing both laboratory and field tests on a number of different inlet configurations. It may be of interest to mention in passing that the hydraulics involved is that taught in elementary courses but that the application requires a thorough knowledge of the conditions under which each of the elementary hydraulic laws are applied.

A great many of the SCS-designed closed conduit spillways consist of a vertical drop inlet from the base of which a barrel carries the flow through the dam. The slope of the barrel may be either steeper or flatter than that of the hydraulic grade line. If the drop inlet is properly proportioned, the spillway will flow full whatever the barrel slope may be even if the outlet is not submerged. Pressures within the barrel will be less than atmospheric and the barrel will lie above the hydraulic grade line if the barrel slope is steeper than the hydraulic grade line. The spillway then acts as a siphon if the broad definition presented earlier is accepted.

A series of tests on drop inlet closed conduit spillways that are pertinent to this discussion was initiated in 1957 at the Urbana Hydraulic Laboratory of the Illinois State Water Survey under the direction of the junior writer. This spillway consists of a circular drop inlet five barrel diameters (5D) high by $5/3$ barrel diameters $(5D/3)$ in diameter. The barrel diameter D is three inches. The 30 per cent slope on which the barrel has been placed initially is considerably steeper than the hydraulic grade line slope. Water enters the drop inlet from the entire periphery of the crest. The head-discharge curve for this spillway has the form shown in Fig. A. Section I of the head-discharge curve represents weir control with the weir at the crest of the drop inlet. Section II is also weir control but air is carried through the spillway with the water. Section III represents pipe control. The head-discharge curve and the flow have been fully described elsewhere.[10]

The primary purpose of the Illinois State Water Survey study is to determine the performance of a flat plate anti-vortex device supported above the crest and to develop design rules for the form and dimensions of the plate. The initial flat plate is two drop inlet (riser) diameters $(2 D_R)$ in diameter with three guide vanes, $120°$ apart, attached to the plate. The guide vanes extend radially outward from the outside edge of the drop inlet. The solid curve of Fig. A is for the case where the anti-vortex plate is located $0.556 D_R$ or higher above the inlet crest, that is, within the range of pipe control represented by Section III. When the plate was lowered into the weir control ranges II or I, it was found that the plate sealed off the inlet as soon as the water level reached the bottom edge of the plate. This prevented access of air to the drop inlet and the flow caused the creation of a partial vacuum. The vacuum sucked air under the plate in sufficient quantity to satisfy the demand. Increasing the water flow caused little or no change in headpool level until the spillway flowed full of water alone. Air was sucked into the spillway in amounts to maintain a vacuum just sufficient to permit the water flow to take place at constant headpool levels. The dash lines in Fig. A show the head-discharge curves for successively lower elevations of the anti-vortex plate. The headpool level in essentially steady, exhibiting none of the fluctuations
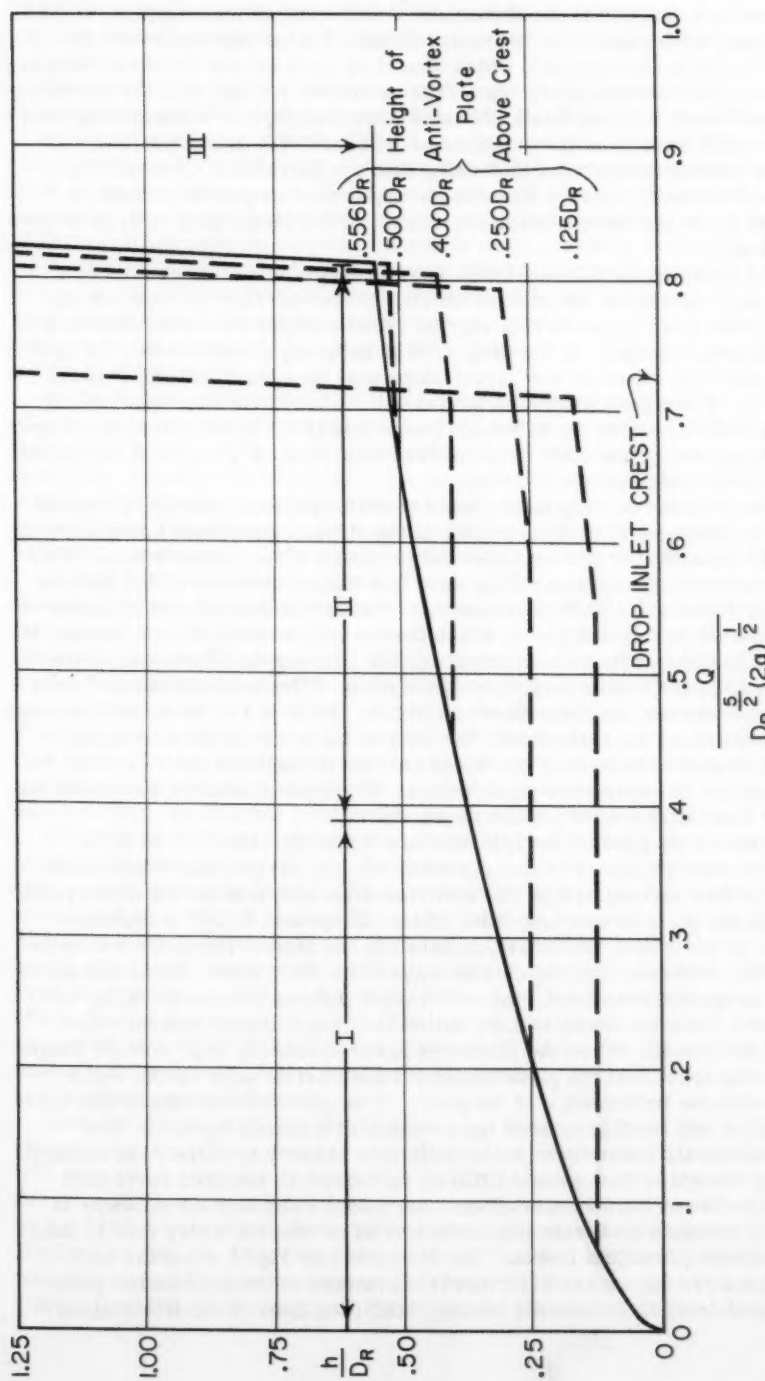
Fig. A - EFFECT OF $2D_R$ CIRCULAR PLATE POSITION ON CLOSED CONDUIT SPILLWAY HEAD-DISCHARGE RELATIONSHIP

shown by Mr. McBirney's Fig. 4. Is this not the partialization and steady headpool level that Mr. McBirney is looking for? The possibility of positively preselecting the priming head within certain presently undefined limits by setting the flat plate at the correct height above the crest should not be overlooked.

To give actual figures for comparison, Mr. McBirney's spillway apparently has a throat area of 1 x 0.5 = 0.5 square feet. The Illinois spillway has a barrel area of $(0.25)^2 \pi/4 = 0.0491$ square feet. The linear multiplication factor to secure comparable priming heads is then $\sqrt{0.5/0.0491} = 3.19$. Plate heights as low as 5/8 inch caused the Illinois spillway to prime. Increasing this to the scale of Mr. McBirney's tests, the priming head becomes 0.17 feet. This corresponds to Mr. McBirney's full flow priming head of 0.52 feet for the standard siphon design and 0.42 feet for the proposed design.

Proper positioning of the flat plate anti-vortex plate used as a siphon primer results in a single-valued head-discharge curve. The curves of two loops shown by Mr. McBirney in Figs. 6 and 11 for the standard and proposed designs do not exist for the flat plate priming device in the positions shown in Fig. A. However, two loops can be obtained for the drop inlet entrance by extending skirts from the periphery of the cover plate down below the crest of the weir. The spillway would then prime on rising stages at the same head as if the skirts had not been used, but the prime would be held until the pool level reached the bottom of the skirts, or to the siphonbreaker if one is used. The skirted arrangement is, of course, what is called a vortex siphon spillway in India.

The priming time, as used by Mr. McBirney for the saddle siphons, does not exist for the drop inlet closed conduit spillway. Priming for the latter spillway takes place gradually as the headpool level rises and does not occur relatively suddenly as for the saddle siphon.

The gradual increase and decrease of flow obtained with the drop inlet siphon also reduces the erosion damage downstream from the outlet which Mr. McBirney says results at many locations from the prime-break-prime cycle of intermittent operation.

The writers do not say that the drop inlet closed conduit spillway, either without or with the flat plate anti-vortex device and primer, is a complete solution to all siphon problems. They do maintain, however, that the potentialities of the drop inlet siphon should not be overlooked.

REFERENCES

1. Joglekar, D. V., Guha, S. K., and Luthra, S. D. L., "High Head Siphons, Their Behavior and Characteristics." Proceedings of the Sixth General Meeting, International Association for Hydraulic Research, The Hague, Vol. 3, pp. C2-1-C2-10, 1955.

2. Rao, N. S. Govinda, "Design of Siphons," Irrigation and Power Journal (India) Vol. XI, No. 4, October 1954.

3. Bradley, Joseph N., "Prototype Behavior," Transactions, ASCE, Vol. 121, 1956, pp. 312-333.

4. Glossary: Sewage and Water Control Engineering, American Society of Civil Engineers, New York, 1949.

5. Rao, N. S. Govinda, "Design of Siphons," Irrigation and Power Journal (India) Vol. XII, No. 4, October 1955.

6. Peterka, Alvin J., "Performance Tests on Prototype and Model," Transactions, ASCE, Vol. 121, 1956, p. 385, Fig. 8.

7. U. S. Dept. of Agriculture, "Agricultural Statistics 1956," Washington, D. C.

8. Blaisdell, Fred W., "Hydraulic Fundamentals of Closed Conduit Spillways," Proceedings-Separate No. 354, ASCE, November 1953.

9. Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Parts II through VII, Results of Tests on Several Forms of the Spillway," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18, Series B, Minneapolis, Minn., March 1958;

   Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Part VIII, Miscellaneous Laboratory Tests," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 19, Series B, Minneapolis, Minn., March 1958; and

   Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Part X, The Hood Inlet," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 20, Series B, Minneapolis, Minn., April 1958.

10. Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Part I, Theory and Its Application," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 12, Series B, Minneapolis, Minn., January 1952, Revised February 1958; and

    Blaisdell, Fred W., "Hydraulics of Closed Conduit Spillways, Parts II through VII, Results of Tests on Several Forms of the Spillway," St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18, Series B, Minneapolis, Minn., March 1958.

SYNTHETIC FLOOD FREQUENCY[a]

Discussion by Max A. Kohler

MAX A. KOHLER,[1] M. ASCE.—Mr. Snyder's approach to generalized, or synthetic, flood frequencies can best be described as an elaboration and improvement of the so-called "rational method." It involves the assumption that the n-year flood can be <u>derived</u> from the n-year rainfall even though the two events do not necessarily bear the relation of cause and effect.

Specifically, the proposed method involves (1) determination of the n-year rainfall for the basin with specified area and concentration time T_c, (2) conversion of n-year rainfall to n-year runoff volume and (3) determination of the flood peak corresponding to the runoff volume. This discussion concerns primarily steps (1) and (2) of the procedure since it is believed that Snyder's or any of several other available techniques are adequate for converting runoff volume to corresponding peak flow.

Before taking up specific aspects of Snyder's procedure, it is perhaps best to consider the basic underlying assumption that a simple relation exists between the n-year flood peak and the n-year rainfall. From a probability point of view, this implies that the composite probability for all factors which affect the relation between rainfall amount and peak flow is unity. Therefore, the only direct means of relation n-year rainfall and flood peaks is through statistical analysis of n-year data. This means that flood-frequency data must be used in deriving the generalized relations. There is no indication on the part of the author that flood-frequency data are necessary or were used. Certainly there was some basis for deciding that the required relation should give "slightly higher runoff than the average" (indicated by average monthly rainfall and runoff). Wouldn't one expect a different rainfall-runoff relation for small basins ($T_c < 4$ hrs) where the major floods result from summer thunderstorms than for the larger basins where such floods result from general rains on wet soil? It is believed the author has tended to account for this variation by an unrealistic reduction of point rainfall for T_c and drainage area. Thus he chose to derive an area-reduction relation from storm data in preference to using one properly derived from frequency data.[1] It is not clear why his results indicate a flatter curve than one based on frequency data, while just the opposite would be expected.

If it is visualized that Snyder's procedure constitutes a method of generalizing, or regionalizing, on the basis of existing flood-frequency data, certain modifications would appear to be appropriate. Although not absolutely necessary, it is suggested that the empirical tie between n-year rainfall and flood-peak be confined to the rainfall-runoff relation. In this case, the n-year

---

a. Proc. Paper 1808, October, 1958, by Franklin F. Snyder.
1. Chf. Research Hydrologist, U. S. Weather Bureau, Washington, D. C.

runoff can be computed from Snyder's relations for all basins with established flood-frequency curves. These values can then be related to corresponding values of n-year basin rainfall derived from the point-rainfall frequency relation (author's Fig. 2) and one of areal rainfall vs duration (T_C) and point rainfall.[1] This is the manner in which the accompanying chart was developed.

The values of peak discharge (Q_p) for various basins and return periods were obtained by Gumbel analysis with appropriate adjustment to give partial-duration data. The average rate of direct runoff (I_r) for time T_C was obtained by use of the author's formula, $I_r = Q_p/500A$, where A is the basin area in square miles.

The first part of the accompanying relation, namely, the area curves in the upper left, was derived from the 10-year rainfall curve of the author's Fig. 2. The 10-year rainfall amounts for various concentration times, or durations, were first plotted and the zero-area curve was drawn through the points. The curves for other sizes of area were then constructed on the basis of the depth-area relation[1] given in U. S. Weather Bureau Technical Paper 29. Although the depth-area relation is based on rather limited data, it is considered to be the best available for the purpose. The second part of the writer's relation, i.e., the return-period curves in the lower left, was obtained by entering the

DATA USED IN ANALYSIS

Basin	Area (sq mi)	T_C (hr)	I_r	T_p (yr) 2 0.001 in./hr	10	100
NW Br. Anacostia R. nr Colesville, Md	21.3	3		138	261	437
Appomattox R. at Farmville, Va	306	31		47	84	139
Codorus Ck. at Spring Grove, Pa.	74.3	4		71	129	214
Goose Ck. nr Leesburg, Va.	338	14		63	150	274
Linganore Ck. nr Frederick, Md.	82.3	7		65	96	142
Monocacy R. at Jug Bridge, Md.	817	24		55	86	131
Octararo Ck. nr Rising Sun, Md.	193	9		72	151	264
Rappahannock R. nr Fredericksburg, Va.	1599	24		48	94	160
Seneca Ck. at Dawsonville, Md	101	7		54	109	189
Slate R. nr Arvonia, Va	235	31		42	72	116
Piney Run nr Sykesville, Md.	11.4	1.5		203	443	791

first quadrant with T_c, going horizontally to the zero-area curve, then downward to the appropriate rainfall intensity (indicated by the author's Fig. 2). Each point so determined was labelled with the corresponding return period and a smooth family of curves were fitted to the points.[2]

It should be pointed out that the return-period curves cannot be drawn to exactly fit areas of all sizes, but the errors are insignificant with respect to the errors of the procedure as a whole. The final curve family (T_c curves) was derived by plotting n-year basin rainfall intensity computed from the first two quadrants against corresponding values of n-year runoff intensity I_r and labeling the points with T_c.

The accompanying table presents the basic data for eleven natural basins in the general vicinity of Washington, D. C., which were used in deriving the T_c-curves. These basins were selected because flood-frequency curves based on long records of observed and synthesized[3] annual peaks were available as were unit hydrographs for estimating T_c.

The application of the writer's relation for estimating n-year runoff intensity is straightforward and needs no further amplification here. The procedures described by Snyder for obtaining T_c and for converting I_r to the n-year peak flow are equally applicable when using the writer's accompanying relation.

In conclusion, the writer wishes to commend the author for presenting new and stimulating ideas on an important phase of hydrologic design.

REFERENCES

1. "Rainfall Intensity-Frequency Regime, Part I: The Ohio Valley," U. S. Weather Bureau Technical Paper No. 29, June 1957.

2. "Hydrology for Engineers", R. K. Linsley, M. A. Kohler and J. L. H. Paulhus, pp. 311-321, McGraw-Hill, 1958.

3. "Flood Frequencies Derived from Rainfall Data," J. L. H. Paulhus and J. F. Miller, Proc. Paper 1451, Jour. Hydraulics Div., ASCE, Vol. 83, December 1957.

DIVINING RODS VERSUS HYDROLOGIC DATA AND RESEARCH[a]

Discussions by Erhard E. Dittbrenner and V. M. Yevdjevich

ERHARD E. DITTBRENNER,[1] M. ASCE.—Mr. Langbein presents a scholar-ly and comprehensive review of the status of the most critical need in hy-drology. It would be difficult to add anything useful to his presentation on hy-drology. However, when he includes "waterwitching" in hydrology—even if only as a strawman to be knocked down and used as a reference—it is time to separate fact from fancy.

Mr. Langbein states: "Modern hydrology offers no support of water witchery." In a certain limited sense this statement may be accurate enough. His further question: ". . . can we be scientific about witchery," is obviously rhetorical, as his implication is that since witchery is perhaps in the realm of fakery, it cannot be scientific.

The answer to his question is, however, an unqualified "Yes." We can be scientific about "witchery." To do so, however, we must go to that portion of physical science which has to do with electrical phenomena. This might seem to take it out of that portion of science which we have artificially labeled "hy-drology." It is not important to argue the point, but it seems pertinent to point out that as we push back the frontiers of our knowledge in our artificially prescribed categories of science it becomes clear that Nature has no such arbitrary divisions. More and more evidence substantiates a belief perhaps more ancient than water witching—that there is a oneness or unity in the Universe.

The writer happens to be a "water witch" or dowzer. Many years ago he was invited to try his hand by a young dowzer who suspected skepticism of the art in my attitude. The dowzer pointed out that many people have the dowzer's capabilities without realizing it. One trial demonstrated the truth of that statement and the basic reason why water witchery has persisted over the world and through the ages. Once a person experiences the phenomena which turns the switch down, he will never be bothered by the scoffing of the "scien-tific" mind. In the writer's case, the switch broke in his hands on his first trial. It has never done so since, but since he never worked at the art very much this may not be very significant.

Since there had to be some scientific explanation for the phenomena, search of literature and query among those more qualified in science than himself finally brought some answers. The "dry" holes can be explained by the knowledge that the forces involved in the phenomena are obviously too small to break the twig, but act on certain muscles which are thereby impelled to bring about the action of the switch without volition on the part of the dowzer.

---

a. Proc. Paper 1809, October, 1958, by W. B. Langbein.
1. Hydr. Engr., U. S. Bur. of Public Rds., Albany, N. Y.

The forces involved vary constantly—during the day, with the weather, and from season to season. Some experienced dowzers are aware of this. Others obviously are not. Since the dowzers are not aware of the source of the phenomena or their powers, they are unable to explain these variations.

Two scientists in England have studied this phenomena extensively. Maby and Franklin, in "The Physics of the Divining Rod," London, 1939, cover the subject comprehensively, but by no means exhaustively. The phenomena has been measured. Instruments have been devised to determine quantitatively the field in which the dowzer works to make his determinations. People have been checked to determine their powers of "divination." About one person in ten has these powers in some degree. Their investigation did not extend to the point of determining the reasons why some do and others do not have them. So far as this writer knows, no one has made any effort in this direction—in fact, since Maby and Franklin's publication, no further scientific literature of that nature on the subject has appeared to the writer's knowledge.

In the writer's own experiences, a routine laboratory experiment during college required the measurement of each student's bodily electrical resistance. His resistance was way out of line with that of any other person in the group. The notes are gone but the memory says it was lower—perhaps a third or a fourth of the "normal." Some interim reading—also not then noted —indicates that persons who become alcoholics must be more careful around electricity than others, as they are more easily electrocuted. There is no implication that persons with low electrical resistance tend toward or become alcoholic victims, but there is an implication that bodily resistance to electric currents vary greatly and these variations have significant physical and physiological results. Medical research some 25 or 30 years ago, has established the fact that the human body is a generator as well as a conductor of electricity. It is logical that generation and restivity would bring about a difference in the electrical field about the human body and its reaction with the field about water "veins."

Here, physiology and the physical sciences appear to have a common interest. So far, nothing has gone beyond what Maby and Franklin have reported, so far as the writer knows. It would seem to be an interesting field for both the medical and the earth scientist to explore.

The question now arises—"so what?" Granted that the dowzer has definite powers, is he useful to the geologist or the hydrologist? Mr. Langbein himself answers that question. In the absence of completely detailed geological and hydrologic subsurface knowledge, the dowzer certainly can be useful. In the writer's opinion, the likelihood of any other knowledge supplanting him appears remote. So far as the writer's knowledge of geology and hydrology permit, neither of the latter branches of science will produce the detailed knowledge available to the informed and experienced dowzer. It seems possible that an extension of the Maby and Franklin research could enhance the dowzer's powers, and mayhap even elevate him to a "professional" category.

This would appear a subject of useful and profitable research. There certainly is no further basis for scoffing.

V. M. YEVDJEVICH,[1] M. ASCE.—Two very interesting problems are discussed by Mr. W. B. Langbein: a) contrast of the divining rods as an

---

1. Hydr. Engr., Foreign senior scientist, National Bureau of Standards, Washington, D. C.

hydrologic decision-maker versus the modern basic-data programs; b) contrast of the increase of gathering basic-data versus the deficiencies in research necessary to guide, analyze and interpret the basic data, and to obtain both the general principles and the regional hydrologic characteristics.

It is clear that modern hydrology does not support "forked stick" procedure which does not have a serious scientific background. But the hydrologist must be aware of some facts which contribute to the persistence of the dowzer's activities, so that the hydrologist may overcome his seemingly apparent disadvantages. Ground water hydrology, frequently very complex, imposes some uncertainty in decisions upon the geohydrologic specialist, while the dowser's short cut way to decisions makes a spectacular impression. The dowser is often a man well acquainted with local conditions, and makes decisions by analogy, based on experiences. He is sometimes a kind of living map, easy for a farmer to consult. He is available, and what is important, cheap. These are the reasons why the use of the divining rod is so persistent and will continue to be so for relatively smaller activities until replaced by good engineering procedures, competitive from other points of view.

The collection of basic data in hydrology has been started and made under some general assumptions for the expected water resources development and control. The basic concepts laid for the collection of hydrologic data depend upon the future water resources development. The first concepts were based on the expected individual and mostly single purpose projects. Next step in development was the multiple purpose use of water resources. The future will probably ask the total and very integrated water resources development in a state of great demands for water control and use. The basic concepts governing the collecting of hydrologic data today should anticipate the situation of water resources development 2 - 3 decades or more from now. This is another reason to support Mr. Langbein's emphasis for a better scientific foundation in general collection and interpretation of basic-data.

When a demand for higher scientific efforts in hydrology is forwarded, the necessary problems connected with it have to be emphasized. The efficiency of the research in hydrology is an important question. Surveying the hydrologic literature of the countries with relatively abundant funds and large number of people employed in research, the impression is that the results do not correspond to the efforts. The quality of the results and the possibilities for the useful applications are rather limited in comparison with the quantity of published papers and other literature. The greatest results for given funds and their best application to the current needs and problems can be achieved only by timely and thorough preparations for expending research.

The timely solution of many problems can help the rapid development of research in hydrology, as for example: the systematic preparation of highly educated engineers and scientists in hydrology; the development of good procedures in putting forward and in selecting the topics to be studied; the exchange of experiences, ideas and new methods; the type of work, in teams, individual or both; etc.

B.
matica
requir
reserv
The

may b
result
To

D_x

I_x,
D_x

S_x

Th
autho
Th
disch
of the
but is
autho

and s

The l
subst

A
ous c

a. P
1. E
E

QUEUING THEORY AND WATER STORAGE[a]

Discussion by B. W. Gould

B. W. GOULD.[1]—The author's paper shows that the techniques of mathematical statistics may be applied in the solution of the problem of determining required water storage, and also in estimating the probable behaviour of a reservoir already constructed.

The service function:

$$D = b + kS$$

may be interpreted in several different ways, each of which gives a different result.

To avoid confusion, the writer has adopted the following notation:

$D_{x, y}$ = Average rate of discharge in the interval between time x and time y.

$I_{x, y}$ = Average rate of inflow in the interval between time x and time y.

D_x = Instantaneous rate of discharge at time x.

S_x = Volume of stored water at time x.

The unit of storage used by the writer is the same as that used by the author, being equal to unit flow for unit time.

The first interpretation of the service function to be considered is that the discharge during an interval of time is determined by the storage at the end of the interval. This type of control would be difficult to achieve in practice, but is considered because it gives the same solution as that obtained by the author.

$$D_{x,x+1} = b + kS_{x+1}$$

and similarly

$$D_{x-1,x} = b + kS_x$$

The basic equations obtained by the author may be verified by subtraction, and substitution of

$$I_{x,x+1} - D_{x,x+1} \quad \text{for} \quad (S_{x+1} - S_x$$

Another interpretation of the service function could be that the instantaneous discharge is dependent on the instantaneous value of storage, i.e.

---

a. Proc. Paper 1811, October, 1958, by W. B. Langbein.

1. Engr., Horsham Waterworks Trust, Vic., Aust., formerly lecturer in Civ. Eng., Univ. of New South Wales, Aust.

$$D_x = b + kS_x$$

and

$$D_{x+1} = b + kS_{x+1}.$$

By subtraction and substitution, this gives

$$D_{x+1} - D_x = k(I_{x,x+1} - D_{x,x+1})$$

If the time interval is sufficiently small, $D_{x, x+1}$ is approximately equal to $1/2 (D_x + D_{x+1})$. Substituting this value and rearranging gives:

$$D_{x+1} = \frac{2k}{2 + k} I_{x,x+1} + \frac{2 - k}{2 + k} D_x$$

instead of the author's

$$D_2 = \frac{k}{1 + k} I_2 + \frac{1}{1 + k} D_1$$

In yet another case, it may be taken that the discharge could be determined by the amount of water in storage at the beginning of an interval. This type of control would be suitable for irrigation of an area containing both semi-permanent (e.g. citrus or vines) and annual (e.g. market garden) development, where a certain minimum amount of water is required to maintain the semi-permanent vegetation, whilst the area of annuals to be planted in any year may be varied according to the amount of stored water available at the beginning of the year. This gives the equations:

$$D_{x,x+1} = b + kS_x$$

and

$$D_{x-1,x} = b + kS_{x-1}$$

Subtracting, and substituting for $(S_x - S_{x-1})$ gives

$$D_{x,x+1} = kI_{x-1,x} + (1 - k)D_{x-1,x} .$$

An estimate of the discharge in the nth year, given by successive substitution of "D" values, is:

$$D_{n-1,n} = k\,I_{n-2,n-1} + (1-k)I_{n-3,n-2} + (1-k)^2 I_{n-4,n-3} +$$

$$\text{etc.} + (1-k)^{(r-1)}I_{n-r-1,n-r} + \text{etc.}$$

From this, the standard deviation of the estimate of the discharge can be calculated as.

$$\sigma_d = \sigma \sqrt{\frac{k}{2 - k}}$$

instead of

$$\sigma_d = \sigma \sqrt{\frac{k}{2 + k}}$$

as given by the author in Eq. (6).

A comparison between the author's Eqs. (12), (13), and (14), and the corresponding equations derived by the writer for the case where the discharge during a period is based on the storage available at the beginning of the period, is given in the following table:

Eqn. No.	Discharge based on final storage.	Discharge based on initial storage.
12	$k = \dfrac{2(\overline{X} - b)^2}{(\overline{X}-m)^2 - (\overline{X}-b)^2}$	$k = \dfrac{2(\overline{X} - b)^2}{(\overline{X}-m)^2 + (\overline{X}-b)^2}$
13	$S = \dfrac{(\overline{X}-m)^2 - (\overline{X}-b)^2}{(\overline{X} - b)}$	$S = \dfrac{(\overline{X}-m)^2 + (\overline{X}-b)^2}{(\overline{X} - b)}$
14	$S = t\dfrac{\sigma^2 - \sigma_d^2}{\sigma_d}$	$S = t\dfrac{\sigma^2 + \sigma_d^2}{\sigma_d}$

It is to be noted that where the discharge is determined by the amount of stored water at the beginning of the period under consideration, the equations give $k = 1$ when $b = m$. In this case, any flow in excess of the minimum flow in any year may be considered as being stored for use in the immediately following year, thus giving time to make plans for its utilization. Under such conditions, the storage required to provide 100 per cent utilization would be the difference between the maximum probable inflow, and the minimum probable natural inflow. This is verified by the equation which gives the required storage as $2t\sigma$.

As in the case considered by the author, the storage required increases rapidly as b approaches $\overline{x}$, and becomes infinite when $b = \overline{X}$.

The accuracy of the "probability routing" methods described by the author in examples 2 and 3 in the paper, is limited only by the number of storage intervals used, the number of successive approximations made, and the ever-present errors of sampling. Electronic computation using standard programmes for the solution of linear simultaneous equations has been used by the writer, who found that smaller intervals of storage volume could be used, and that the need for successive approximations was eliminated because the solution was provided in a direct manner.

HY4

Your attention is invited

NEWS
OF THE
HYDRAULICS
DIVISION
OF
ASCE

AMERICAN
SOCIETY OF
CIVIL
ENGINEERS
FOUNDED
1852

DIVISION ACTIVITIES
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

April, 1959

PURPOSE OF THE HYDRAULICS DIVISION
(Quoted from the Official Register)

"The advancement and dissemination of knowledge relating to the occurrence of water in nature and its behavior in structures, water courses, and underground.

"In particular the field of the Hydraulics Division shall embrace meteorology and hydrology as they affect the engineer, fluid mechanics in engineering usage, and applied hydraulics as a branch of engineering science which furnishes the basis for hydraulic design and for the practical use of water in the different specialized branches of hydraulic engineering."

COMMITTEES SPONSOR FOUR SESSIONS AT CLEVELAND

Four committees of the Division have each prepared a session for the ASCE Convention at Cleveland on May 4-8, 1959. The four sessions were published as tentative in the February Newsletter, and the latest information indicates that no changes have been made to the programs.

HYDRAULICS CONFERENCE AT FORT COLLINS, COLORADO
July 1-3, 1959

The eighth national Conference of the Hydraulics Division will be held at Fort Collins, Colorado on July 1, 2, 3. The hosts this year are the Colorado and Wyoming Sections of ASCE and Colorado State University. There will be three days of technical sessions and one day of an extensive field trip through the beautiful Colorado Rockies. In addition to the professional sessions there will be social sessions in which fun and good fellowship can be enjoyed by all.

Conference Headquarters on Colorado State University Campus

Aylesworth Hall, a beautiful new dormitory on the University Campus, will be the housing center for the conference. This is close to the new Engineering

---

Auditorium where all conference sessions will be held. Rooms will be available at the following special rates: $2.50 single, $2.00 per person double. Except for the special events, meals will be available at the following prices:

	Adults	Children
Breakfast	$0.75	$0.65
Lunch	1.00	0.95
Dinner	1.25	1.10

In addition to the facilities listed above, there are several hotels and motels available. Write to the Housing Committee if alternate accommodations are preferred.

Facilities for Technical Sessions and Tours

The auditorium in the new Engineering Center has been reserved for the technical sessions. A conducted tour of the Hydraulics Laboratory and other engineering facilities is one of the features of the program.

A guided tour of the Big Thompson Irrigation and Power Project has been planned. This trip includes a ride across the Continental Divide over Trail Ridge Road (elevation 12,000 ft) in scenic Rocky Mountain National Park. A trip through the Hydraulics Laboratory of the Bureau of Reclamation in Denver is also in the schedule.

Social Events

After the field trip on Tuesday, June 30, a social hour with cocktails has been planned in the evening. A barbecue will be held on Wednesday evening and a special banquet on Thursday evening. All of these events are open for family participation.

Additional activities are being planned for the ladies and children in attendance at the conference.

Vacation Possibilities

Fort Collins is located near the heart of scenic Colorado. Bring the family and plan to spend a vacation in the Rockies where the scenery is magnificent, the fishing is good, and many of the housing facilities include swimming pools and other recreational facilities. Hiking, horseback riding, water skiing, tennis, and other facilities are available for families who enjoy contact with the great outdoors. Since normal tourist demands are heavy during the summer months, arrangements for a vacation in the mountains should be made early through the Housing Committee of the Conference.

Six Committees Sponsor Technical Sessions

The tentative program for the Hydraulics Division conference is as follows:

Sponsored by the Hydraulic Structures Committee
    Presiding: C. E. Kindsvater, Chairman, Executive Committee, and
                   A. J. Peterka, Member, Hydraulic Structures Committee
    THE MANIFOLD STILLING BASIN, Maurice L. Albertson, Director, Research Foundation, Colorado State University, Fort Collins, Colo., and Gene R. Fiala, Highway Design Engineer, U. S. Bureau of Public Roads, Portland, Ore.

THE VORTEX CHAMBER AS AN AUTOMATIC FLOW-CONTROL DEVICE, Richard C. Kolf, Assistant Professor, and Paul B. Zielinski, Assistant Instructor, Marquette University, Milwaukee, Wis.

NEW CONCEPTS IN TUNNEL SPILLWAY DEFLECTOR-BUCKET DESIGN, T. J. Rhone and A. J. Peterka, Engineer, Bureau of Reclamation, Denver, Colo.

Sponsored by the Hydromechanics Committee
    Presiding: Arthur T. Ippen, Vice-Chairman, Executive Committee, and
                Donald R. F. Harleman, Member, Hydromechanics Committee

SYMPOSIUM ON HYDRAULIC MODELS—PART ONE

A UNIFIED CONCEPT OF DYNAMIC SIMILARITY IN FLUID MODELS, Donald R. F. Harleman, Associate Professor of Hydraulics, Massachusetts Institute of Technology, Cambridge, Mass.

SIGNIFICANCE AND APPLICATION OF FROUDE AND REYNOLDS NUMBERS AS CRITERIA FOR SIMILITUDE, H. K. Liu, Associate Civil Engineer, and M. L. Albertson, Director, Research Foundation, Colorado State University, Fort Collins, Colo.

DYNAMIC SIMILARITY IN WAVE AND TIDAL MODELS, F. Gerritsen, Associate Professor, Coastal Engineering Laboratory, University of Florida, Gainesville, Fla.

SEPARATION AND SCALING OF INERTIA AND VISCOUS FORCES IN PERIODIC FLOWS, Gershon Kulin, Hydraulic Engineer, Fluid Mechanics Section, National Bureau of Standards, Washington, D. C.

Sponsored by the Tidal Hydraulics Committee
    Presiding: J. B. Tiffany, Secretary, Executive Committee, and
                L. P. Disney, Chairman, Tidal Hydraulics Committee

SYMPOSIUM ON INSTRUMENTATION

NEW INSTRUMENTS DEVELOPED BY THE U.S.G.S. FOR THE MEASURE-MENT OF TIDAL FLOW, E. G. Barron, Hydraulic Engineer, Research Section, Surface Water Branch, U. S. Geological Survey, Columbus, Ohio.

RECENT DEVELOPMENTS BY THE COAST AND GEODETIC SURVEY OF EQUIPMENT FOR THE MEASUREMENT AND REMOTE RECORDING OF TIDES AND CURRENTS, A. J. Goodheart, Division of Tides and Currents, U. S. Coast and Geodetic Survey.

A PRESSURE-TYPE TIDE AND WAVE RECORDER, E. H. Bowler, Instruments and Control Systems Laboratory, National Research Council of Canada.

SPECIAL DEVELOPMENTS IN OCEANOGRAPHIC INSTRUMENTS, J. M. Snodgrass, Head, Special Developments Division, Scripps Institution of Oceanography.

Sponsored by the Sedimentation Committee
    Presiding: Harold M. Martin, Member, Executive Committee, and
                E. J. Carlson, Member, Sedimentation Committee

SEDIMENTATION AND CONSERVATION ON WESTERN ARID LANDS, H. V. Peterson, Branch Area Chief, General Hydrology Branch, Division of Water Resources, U. S. Geological Survey.

THE USE OF ULTRASONICS IN THE MEASUREMENT OF SUSPENDED SEDIMENT SIZE DISTRIBUTION AND CONCENTRATION, Gordon H. Flammer, Assistant Professor of Civil and Irrigation Engineering, Utah State University.

Sponsored by the Hydrology Committee
    Presiding: Maurice L. Dickinson, Member, Executive Committee, and
                    V. A. Koelzer, Member, Hydrology Committee

SYMPOSIUM ON THE USE OF ELECTRONIC COMPUTERS IN HYDROLOGY

APPLICATION OF ELECTRONIC COMPUTERS TO THE SOLUTION OF HYDROLOGIC PROBLEMS IN THE BUREAU OF RECLAMATION, Francis E. Swain and H. S. Riesbol, Bureau of Reclamation, Denver, Colo.

THE ROLE OF ELECTRONIC COMPUTERS IN HYDROLOGIC STUDIES OF THE TVA, W. M. Snyder, Tennessee Valley Authority, Knoxville, Tenn.

COLUMBUS RIVER SYSTEM POWER ANALYSIS BY USE OF DIGITAL COMPUTER, David J. Lewis and Loren A. Shoemaker, Corps of Engineers, U. S. Army, North Pacific Division, Portland, Ore.

THE USE OF HIGH-SPEED COMPUTING EQUIPMENT IN WATER RE- SOURCES INVESTIGATIONS OF THE U. S. GEOLOGICAL SURVEY, G. E. Harbeck, Jr., and W. L. Isherwood, U. S. Geological Survey, Denver, Colo.

Sponsored by the Flood Control Committee
    Presiding: A. R. Chamberlain, Co-Chairman, Fort Collins Hydraulics
                    Conference, and Arno Lenz, Member, Flood Control Committee

FLOOD CONTROL PROBLEMS IN THE SOUTH PLATTE RIVER BASIN, COLORADO, Stanley A. Miller, Supervisory Water Conservation Project Engi- neer, Colorado Water Conservation Board, Denver, Colo.

CHANGES IN URBAN OCCUPANCE OF FLOOD PLAINS IN THE UNITED STATES, Gilbert F. White, Chairman, Department of Geography, University of Chicago, Chicago, Ill.

A NEW APPROACH TO LOCAL FLOOD PROBLEMS, Gen. Herbert D. Vogel, Chairman of the Board, Tennessee Valley Authority, Knoxville, Tenn.

A SUGGESTED OUTLINE OF LEGISLATION TO ACCOMPLISH FLOOD PLAIN ZONING, Author to be announced.

HYDRAULICS DIVISION MEMBERS SERVE IN THE INTERNATIONAL STANDARDS ORGANIZATION

Announcement has been made by ASCE Headquarters of the appointments of Messrs. Rolland W. Carter, Frank B. Campbell, and Norbert H. Leupold as ASCE representatives to Subcommittee ISO/TC 30/SC1 of the International Standards Organization, with subject: Flow Measurements in Open Channels.

Mr. Carter will serve with Work Group No. 1, which deals with velocity- area method of measurements; Mr. Leupold with Work Group No. 2 will be concerned with notches, weirs and flumes; and Mr. Campbell will serve with Work Group No. 3 regarding glossary of terms.

A meeting of the Subcommittee was held in London, December 8-15, 1958, attended by Mr. Carter who was the representative of the Hydraulics Division, ASCE and the American Standards Association.

REVIEWS OF TECHNICAL PAPERS ARE INADEQUATE

The Hydraulics Division during the past three or four years has seen a gradual increase in the number of papers received for review and consider- ation for publication. This increase has reached the point where adequate

reviews by members of the Division are not accomplished in a great many
instances. As the result a number of papers are recommended for publi-
cation without a critical review to determine their real importance. The
problem stems not only from the increase in the number of papers but also
from a lack of information as to who is most qualified to review a paper and
the time required for the reviewing process. Also, the problem is further
complicated when the reviewer and/or the author are not fully aware of the
standards and procedures established for processing technical papers.

The Division Executive Committee in its meeting last November appointed
a task force composed of the present and past chairman of the Division's
Publication Committee (Messrs. J. W. Ball, E. P. Fortson, Jr., and H. G.
Dewey, Jr.) for the specific purpose of studying the technical paper review
procedures with a view to improving and accelerating the process. The suc-
cess of their efforts must ultimately depend upon an understanding of the
publication procedures by both authors and reviewers and an increased inter-
est in resolving the problem by all members of the Division. We can start
now—see "We Can Improve the Processing of Technical Papers," and com-
plete the form on the inside back cover of this Newsletter.

WE CAN IMPROVE THE PROCESSING OF TECHNICAL PAPERS

Do you know how much work goes into the processing of your technical
papers before they appear in the Journal? No doubt you have pondered this
question and wondered about the answer. The following brief outline and
information are offered to give you at least a partial answer to the question,
and to provide you the opportunity to help in speeding up and improving the
process:

1. Two copies of your paper are sent from the New York Office to the
   Publications Chairman of the Division for review, that is, if the re-
   quired three copies are sent to the New York Office, as instructed on
   the inside front cover of each issue of the Journal.
2. The two copies received by the Publications Chairman are sent to two
   reviewers, experts in the field covered by the paper, insofar as the
   Chairman knows. The reviewers are selected from a list of some 300
   names collected over the years by the chairmen of the Committee. Ad-
   ditional competent reviewers are needed.
3. The reviewers make a detailed study of the paper as to its technical
   contents, adequacy and suitability for publication in the Journal and re-
   port their findings to the Chairman.
4. From the reviewers' reports the Chairman decides whether the paper
   is to be reviewed a third time, published, or declined. At least two
   unfavorable reports are required to decline publication. The paper is
   either accepted as presented, accepted on condition that changes are
   made to clarify or satisfy questions and requirements, or declined.
   Once all the questions and requirements for a paper have been satisfied,
   the paper is sent to the New York Office with the appropriate recom-
   mendation.

The work outlined above is all done voluntarily and without remuneration.
At times the regular work of those involved prevents immediate handling. It
may take as much as three weeks (more under special circumstances) for the

Chairman to record receipt of a paper, choose reviewers and get copies of the paper to the reviewers. It may require as much as three months for a reviewer to make his review and report to the Chairman, depending on his work load. (He is asked to do it within a month, if possible.) The review process can be quite lengthy, particularly in cases where only one copy of the paper is furnished. When extensive changes are required before publication can be recommended, the time taken by the author to do this work may vary up to 6 months, or longer, depending on the time the author is willing or able to spend on the paper and the extent of the revisions required.

You, as authors, reviewers, and prospective reviewers can speed up and improve this process by, (a) preparing your paper to meet all the requirements listed on the inside front cover of each issue of the Journal; (b) by signifying your willingness to review papers in your fields of interest (see form on back cover); (c) by making prompt reviews of papers sent to you for review; and (d) by making, expeditiously, changes or comments to clarify questions and requirements brought out by the review.

HYDRAULICS DIVISION ACTIVITIES DUE FOR FURTHER INCREASE

The Division Newsletter of October 1958 describes briefly the report made in March by the ASCE Task Committee on Water Resources. As recommended in the report, an ASCE Water Resources Coordinating Committee was appointed in December 1958. Those appointed to the Committee are: Finley B. Laverty, Chairman, Joseph B. Tiffany, representing the Hydraulics Division, William W. Donnan, the Irrigation and Drainage Division, George J. Vencill, the Power Division, Richard R. Kennedy, the Sanitary Engineering Division, and Roger H. Gilman, the Waterways and Harbors Division. Don P. Reynolds was assigned as committee secretary.

The purpose of the Coordinating Committee is to coordinate and supervise the new organization and activities which are expected to result from the very significant study made by the Task Committee on Water Resources. As the next step in its activities the Coordinating Committee has asked each participating Division to undertake study of its own organization, as related to the recommendations of the Task Committee, and to furnish recommendations to the Coordinating Committee. Suggestions to the Executive Committee, Hydraulic Division, include a request for specific proposals for additional committees and personnel to man these committees. A preliminary report from the Executive Committee was due in early March 1959.

The actions proposed in the report of the Task Committee indicate that if implemented the Hydraulics Division is the one most directly affected in the field of water resources. The new programs and studies that would become the responsibility of the Hydraulic Division would be so extensive and important as to approximately equal all other activities of the Division.

IMPROVEMENT IN PUBLICATIONS STANDARDS URGENTLY NEEDED

The minutes of various committee meetings as well as correspondence between individuals dealing with ASCE publications contain many references to the current lack of proper recognition given to the Division Journals of ASCE by prospective authors, readers, and library officials. The Journals in their present form are looked upon as an interim measure pending publication of

some of the papers in ASCE Transactions. Thus, the general membership is deprived of complete coverage of some of the best papers of direct interest to them. Authors, having no assurance that their work will appear later in ASCE Transactions, naturally give preference to journals or other recognized publications of other professional societies. In this manner some outstanding papers written by ASCE members are lost insofar as the records and the history of the Society are concerned. Moreover, this lack of recognition of the Journals not only affects the attitude of the author toward producing a worthwhile paper, but also the attitudes of those individuals who are asked to review the paper as well as those who participate in discussions of the techni-cal material.

It appears therefore that action to improve the standards of publication for the Journals is urgently needed. To this end suggestions have been made to the Publications Committee, ASCE, that early attention be given to converting the Journals into suitable and attractive media for publication of professional papers, including the adoption of publication standards that will be recognized by the Library of Congress and other scientific libraries as being appropriate for permanent inclusion in their collections.

AN INTERESTING PAPER ON HYDROLOGY PUBLISHED IN JOURNAL OF THE SANITARY ENGINEERING DIVISION

The paper "Hydrology of Urban Runoff" by A. L. Tholin and C. J. Keifer is being published in the March Journal of the Sanitary Engineering Division as Proceedings Paper No. 1984. Members who are interested in obtaining copies of this paper can order it by using the coupon in the "Proceedings Available" section of the March issue of CIVIL ENGINEERING.

FOR YOUR CALENDAR

ASCE Meetings

May 4-8, 1959	ASCE, Cleveland Convention
July 1-3, 1959	Hydraulics Division, Fort Collins Conference
October 19-23, 1959	ASCE, Washington, D. C., Convention
March 7-11, 1960	ASCE, New Orleans Convention
June 19-23, 1960	ASCE, Reno Convention
October 9-13, 1960	ASCE, Boston Convention
April 10-15, 1961	ASCE, Phoenix Convention
October 16-20, 1961	ASCE, New York Convention
February 1962	ASCE, Houston Convention
May 1962	ASCE, Omaha Convention
October 15-19, 1962	ASCE, Detroit Convention

Non-ASCE Meetings

April 13-15, 1959	ASME Hydraulic Conference, University of Michigan
June 15-19, 1959	American Society for Engineering Education, Pittsburgh, Pennsylvania

June 17-20, 1959   National Society of Professional Engi-
                              neers, New York City
June 18-20, 1959   ASME Applied Mechanics Conference
                              Virginia Polytechnical Institute,
                              Blacksburg
August 24-29, 1959   International Association for Hydraulic
                              Research, Montreal, Canada
September 9-11, 1959   Midwestern Conference on Fluid and
                              Solid Mechanics, University of
                              Texas, Austin

Deadline dates for Newsletter contributions: June issue - April 15.
                                              August issue - June 15.

USE THE HYDRAULICS DIVISION NEWSLETTER

You are urged to continue use of the Division Newsletter for announce-
ments, inquiries, personnel news, committee reports, surveys and other items
of interest to Division members. A short note covering the highlights of com-
mittee meetings (including task forces) is particularly requested.

                              GUY L. ARBUTHNOT, Jr.
                              P. O. Box 631
                              Vicksburg, Mississippi
                              Newsletter Editor

P.S. Please fill out the form on the next page now and return it to Mr. Ball.

REVIEWERS NEEDED

Your help is earnestly solicited in obtaining willing and qualified members who are experts in their fields to review technical papers. The form below is furnished for your convenience in nominating yourself, and/or others for this important work.

Reviewers are needed in the following fields:

1. Research	15. Tides and Waves
2. Fluid Mechanics	16. Beaches and Estuaries
3. Hydraulic Models	17. Ice and Permafrost
4. Open Channel Flow	18. Water Measurement
5. Closed Conduit Flow	19. Design
6. Turbulence	20. Spillways
7. Density Currents	21. Outlet Works
8. Cavitation	22. Canal Structures
9. Sedimentation	23. Gates and Valves
10. Hydrology	24. Operation and Maintenance
11. Floods	25. Analogs
12. Ground Water	26. Instrumentation
13. Water Supply	27. Other _____
14. Meteorology	

Your name and address: Indicate fields of interest by number, thusly: (1), (2), etc.:

Name and address of recommended reviewers: Indicate Reviewer's fields of interest by number:

Mail to:
James W. Ball, Chairman
Committee on Publications
Hydraulics Division, ASCE
Bureau of Reclamation
Building 56, Denver Federal Center
Denver 25, Colorado